WHITLOCK DALRYMPLE POSTON & ASSOCIATES, INC.
CONSULTING ENGINEERS

EVALUATION OF SILVER SPRING TRANSIT CENTER

SILVER SPRING, MD

WDP Project No. 12421

May 2, 2013

Prepared for:

Washington Metropolitan Area Transit Authority
Office of Track Structures & Facility Engineering
Department of Transit Infrastructure and Engineering Services

3500 Pennsy Drive
Building C, Room C144
Hyattsville, Maryland 20785

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Respectfully submitted,

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EXECUTIVE SUMMARY

An evaluation of the Silver Spring Transit Center has been completed by WDP & Associates, Inc. (WDP). The Silver Spring Transit Center (SSTC) is a newly constructed multi-modal transportation facility located in Silver Spring, MD. The goal of the evaluation by WDP was to assess both the structural design and expected long-term durability of the structure prior to the acceptance of the structure into the Washington Metropolitan Area Transit Authority (WMATA) inventory. The evaluation was motivated by the observation of concrete cracking and questions regarding the thickness of the concrete slabs in the elevated portions of the structure. The evaluation by WDP indicated that significant design and construction deficiencies have resulted in a structure that will be unable to achieve the 50 year design service life specified in the WMATA design requirements without significant repairs and a long-term maintenance program to address durability problems.

The evaluation by WDP indicated that the overall design of the SSTC was deficient due to the restrained nature of the structure and the inability of the structure to accommodate normal thermal movements. The restrained nature of the as-designed structure was a significant factor in the observed cracking at the structure. Structural analyses indicated that the as-built structure will have adequate ultimate strength to resist design loads using the design concrete strength.

Significant construction deficiencies were also observed in the evaluation. The construction deficiencies include omission of post-tensioning tendons in the pour strips on Level 330, thin concrete slabs, extensive cracking on the elevated slab surfaces as a result of restrained shrinkage and finishing problems, exposed and low concrete cover to post-tensioning tendon ducts at numerous locations, and low entrained air content in the top surface of the elevated slabs.

Attempts to repair the cracking in the elevated slabs have been made by the contractor. To date, the repairs have not been effective in controlling the movement of water through the slabs. Further, the extent of cracking has increased over time, through the formation of new cracks and the extension of existing cracks. The cracking in the elevated decks is undesirable as the cracks allow the ingress of water and other deleterious materials into the concrete slabs.

To achieve the design service life, a long-term protection strategy will be required to enhance the durability of the structure. The primary goal of the protection strategy is to prevent the ingress of water and deicing salts into the elevated slab surfaces. The long-term protection strategy is in addition to the repair of the construction defects that are present in the structure. Different materials, with different expected service lives, can be used to protect the elevated slabs surfaces. The life cycle costs for repair and maintenance of the SSTC structure will be significantly increased as a result of the design and construction defects.

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1.0 INTRODUCTION

Based upon a proposal dated July 24, 2012, WDP & Associates, P.C. (WDP) was retained by the Washington Metropolitan Area Transit Authority Office of Track Structures and Facility Engineering (WMATA) to evaluate the Silver Spring Transit Center (SSTC), located in Silver Spring, MD. The SSTC was developed by the Montgomery County Maryland Department of General Services to increase the efficiency of public transportation in the Silver Spring, MD area. Funding for the project was obtained from the Federal Transit Administration, Montgomery County, MD and the Maryland Mass Transit Administration. After completion of construction, the SSTC is intended to be operated and maintained by WMATA. The SSTC was intended to have a minimum design service life of 50 years.

The evaluation of the SSTC by WDP was motivated by the observation of cracking in the post-tensioned beams and slabs and concerns about the effects of "thin" concrete slab members. The evaluation by WDP included a limited field investigation to examine the as-built construction, a review of documents related to the design and construction, and structural analyses to examine the as-designed and as-built adequacy of the structure based on applicable codes and design load requirements. WDP also reviewed the repairs recommended by other parties. None of the repair options developed by other parties have been approved or accepted by WMATA.

The following sections describe the results of field testing and structural analyses of the structure by WDP.

2.0 BACKGROUND INFORMATION

The Silver Spring Transit Center (SSTC) was developed to be a multi-modal transportation hub for the Silver Spring, MD area. The structure was designed as a cast-in-place concrete structure with bonded post-tensioning used in the beam and slab construction. The structure consists of two elevated levels and a slab-on-ground. The upper elevation (Level 350) was designed for vehicular (taxi and passenger drop-off), with the lower levels (Level 330) and the slab-on-ground designed for bus transit. Figures 2-1 and 2-2 show representative views of the exterior of the structure in March 2012.

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Major participants in the design and construction process included:

- Parson Brinkerhoff (PB) – engineer of record.
- Foulger-Pratt Contracting, LLC – general contractor.
- Facchina Construction Company – concrete sub-contractor.
- The Robert Balter Company – special inspection agency.

7Construction began in 2009 and is still in progress at the time of the preparation of this report.

Figure 2-1. Exterior of SSTC in March 2012.

Figure 2-2. Interior of SSTC in March 2012. Soffit of Level 350 shown from Level 330.

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2.1 DOCUMENTS REVIEWED

To assist in the evaluation of the SSTC, WDP was provided with a series of documents related to the design and construction at the SSTC. These documents included the plans and specifications used in the design, project specific design criteria and documents prepared in response to the purported deficiencies at the SSTC.

2.1.1 Design and Construction Documents

The design documents reviewed by WDP included:

- Design drawings for Silver Spring Transit prepared by Parsons Brinkerhoff (PB) – ASI Compilation Set dated 3/4/11.
- Project Specifications for Silver Spring Transit Center (Vol. 2 and 3) prepared by PB.

Construction documents reviewed by WDP included:

- RFI's.
- VSL Shop Drawings.
- Special Inspection Reports.
- Post-tensioning Stressing Records.

2.1.2 Design Loads and Criteria

The design criteria for the SSTC were shown in the design drawings to include the following:

- 2003 International Building Code.
- WMATA Manual of Design Criteria – Release 6.
- AISC Specifications for Design, Fabrication and Erection of Structural Steel for Buildings, Latest Edition.
- ACI 117 – Standard Tolerances for Concrete Construction and Materials.
- ACI 301 – Specifications for Structural Concrete for Buildings.
- ACI 302.1R – Guide for Floor and Slab Construction.
- ACI 304 – Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete.
- ACI 305R – Hot Weather Concreting.

- ACI 306R – Cold Weather Concreting.
- ACI 308 – Standard Practice for Curing Concrete.
- ACI 318 – Building Code Requirements for Structural Concrete.
- CRSI MSP 2 – Manual of Standard Practice for Reinforced Concrete Construction.
- Post-Tensioning Institute – Specifications for Bonded Single Strand or Multi-Strand Tendons for use in Corrosive Environments.

The design loads included:

- Applicable dead loads.
- Superimposed dead load of 35 psf in drive lanes for future wearing surface.
- HS 25-44 Design vehicle in drive lanes with 33% impact factor.
- Live Load = 150 psf in pedestrian / sidewalk areas.
- Snow loads (open areas) based upon P_g = 30 psf.
- Wind and seismic loads.

The WMATA Manual of Design Criteria contained specific requirements for the design of parking facilities using post-tensioned concrete. These include:

- Design for a minimum 50 year service life.
- Limit extreme fiber stresses in tension to $6\sqrt{f'_c}$ under service load levels.
- Limit extreme fiber stresses in tension to $6\sqrt{f'_{ci}}$ at transfer even if bonded auxiliary reinforcement is provided.
- Concrete materials shall have a minimum compressive strength of 6,000 psi at 28 days, with a maximum water / cement ratio of 0.38. Concrete shall also contain a calcium nitrite based corrosion inhibitor.

The WMATA design criteria are generally more restrictive than the design criteria presented in the current version of ACI 318. The limits on tensile stresses in service and at transfer are intended to produce an "uncracked" structure under service load conditions. The more stringent design criteria by WMATA are intended to create a more durable structure. These additional requirements are acceptable in new construction as the ACI 318 code only presents minimum requirements.

Based upon the specification of ACI 117 for construction tolerances, the following are some of the allowable tolerances for slab thickness and reinforcement placement:

- Thickness of suspended (elevated) slabs: - ¼ in.
- Nonprestressed reinforcement cover from formed surface: +/- 3/8 in.
- Nonprestressed reinforcement cover from top surface: - 3/8 in.

- Vertical placement of prestressing ducts: +/- 3/8 in.

Variations in thickness or reinforcement placement that exceed these values would be deemed not satisfy the project specifications.

2.2 DOCUMENTS PREPARED BY OTHER PARTIES

In addition to the evaluation by WDP, several other parties have developed reports and other documents on the SSTC. The documents reviewed by WDP included:

- Report prepared by CTL Group dated March 2, 2012. Report prepared for Foulger Pratt.
- Report prepared by SGH dated March 14, 2012. Report prepared for Shapiro Lifschitz & Schram, P.C.
- Engineer's Report prepared by Desman Associates dated June 11, 2012. Report prepared for Maryland Transit Administration.
- Reports prepared by PB related to the evaluation of the extent of "thin slab" construction and the cracking at the SSTC (various report dates, primary date February 27, 2012) and a report (dated May 22, 2012) that reviewed the results presented in the SGH report and included recommendations for the installation of a latex modified concrete overlay on the SSTC drive areas.
- Proposed Level 330 pour strip strengthening plan dated November 27, 2012. Prepared by STRUCTURAL Technologies for Facchina Construction Company.

These reports were reviewed to evaluate the previous assessments completed at the SSTC. Overall, the reports describe efforts by the various parties to determine the impact of the various deficiencies on the structural capacity and long-term durability of the structure. Specific comments on the reports are presented in the following sections.

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2.3 COMMENTS ON PREVIOUS REPORTS

2.3.1 Comments on CTL Group Report

The report by CTL Group (CTL) described the petrographic evaluation of 13 core samples removed from the SSTC. The goal of the petrographic analyses by CTL was to determine if an evaporation retarder / finishing aid (Eucobar) used in the slab construction was contributing to the observed cracking on the slab surfaces and if the long-term durability of the slabs was reduced by the Eucobar material.

The report by CTL concluded that the Eucobar material did not result in any damage to the slab surfaces or impact the long-term durability of the slab surfaces. In the course of the petrographic evaluation, CTL estimated (in lieu of a more accurate point count measurement) the air content of the core samples. The report indicated the entrained air content and air void space in the top surface of the cores (0 to 30 mm from the top surface) was likely to be insufficient for long-term freeze thaw durability and deterioration in the form of scaling was possible in saturated areas exposed to freezing conditions. The report indicated that a silane / siloxane based surface sealer was to be applied to the concrete surfaces to improve the durability of the concrete and indicated the sealer application was appropriate.

2.3.2 Comments on SGH Report

The investigation by SGH included surface penetrating radar (SPR) to examine the thickness of the post-tensioned slabs, structural analyses to examine the as-built capacity of the structure and a review of the expected durability of structure using the results of petrographic results presented in the CTL report. The SGH report also included an estimation of the expected service life of the structure prepared by Tourney Consulting Group (Tourney).

Based upon the results of structural analyses, SGH concluded the as-constructed SSTC had sufficient capacity to resist design loads. The conclusions in the SGH report were based upon several assumptions regarding the concrete strength, load factors and allowable tension stress. These included:

- The use of provisions ACI Chapter 20 to allow for an increase in phi factors based upon the assumption that the SSTC is an existing structure. The SSTC does not satisfy the definition of an existing structure in the 2003 International Building Code (IBC). The 2003 IBC defines an existing structure as a building or structure for which a legal building permit and a certificate of occupancy have been issued. Therefore based upon the incomplete nature of the construction and the lack of a certificate of occupancy, the use of these provisions is not appropriate. Further, the commentary to Section 20.1.2 of ACI 318-05 indicates the analytical investigation be based upon the "properties of the materials in place."
- The concrete strength used in the SGH analyses was based upon a 56 day compressive strength of laboratory-cured specimens. The compressive strength value was not confirmed by testing of core specimens removed from the structure, and therefore it may not be representative of the in-place concrete strength

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- The SGH analyses used an allowable concrete tensile stress of 7.5 $\sqrt{f'_c}$, which is allowable by ACI 318, but is in excess of the more restrictive value used in the WMATA Manual of Design Criteria

The report by Tourney involved prediction of the estimated time to the onset of reinforcing steel corrosion based upon the concrete material properties, in-situ concrete, planned surface treatments and exposure conditions. Tourney assumed that a silane / siloxane sealer would be applied at 10 year intervals. Based upon these assumptions, Tourney predicted the service life of the SSTC to be in excess of 100 years. The Tourney predictions are limited by several factors, including:

- The concrete material properties used in the model were based upon assumed material properties and not from tests on the actual concrete materials used in the SSTC construction
- The durability model does not account for the extensive slab cracking. The cracking will increase the transport of chlorides into the concrete
- The durability model did not consider the potential for freeze-thaw or scaling damage resulting from the low entrained air content in the top surface of the concrete

2.3.3 Comments on Desman Report

The report by Desman did not include results of any testing or additional analyses. The report recommended the installation of a supplemental wearing course to provide additional protection to reinforcing steel and limit the ingress of water through the slabs. Two different materials were recommended, a two in. thick rubberized asphalt wearing surface, or a thin (1/4 inch nominal thickness) polymer-based overlay system developed for bridge decks.

The concept of a thin polymer overlay has merit, as these materials have been used on numerous bridges. However, cracks may reflect through the overlay over time. The rubberized asphalt is not recommended as they are unlikely to have the required durability and the two inch design thickness is not desirable.

2.3.4 Comments on PB Reports

The February 27, 2012 report by PB presented the results of a laser scanning and SPR investigation to identify locations on Levels 330 and 350 where thin concrete slabs were present. The report included results of a laser thickness scan (performed by Greenhorne & O'Mara, Inc.) to quantify the thickness of the elevated slabs on Level 330 and 350. Based upon these results, additional structural analyses were performed to assess the effect of the "thin" slabs on the expected performance of the structure. The analysis by PB was based upon consideration of "design strips" in lieu of a full model of the structure. The analysis did not consider use of Chapter 20 in ACI 318. The report did not indicate if moment redistribution was used in the analysis. The re-analysis by PB included the effective prestressing force, as determined by the construction stressing records, to determine if allowable stress limits were exceeded and if ultimate strength requirements were satisfied. The report concluded that:

- Based upon the results of the laser slab thickness survey, slab sections on both Level 330 and 350 are thinner than allowable by the project specifications.
- On Level 330, in the thin slab areas, the service load stresses were greater than the 6 $\sqrt{f'_c}$ limit in the design requirements.
- On Level 330, in the thin slab areas, the flexural capacity of the slabs was exceeded in some locations.
- On Level 330, the capacity of the beams in the thin slab areas was determined to be adequate.
- On Level 350, in the thin slab areas, the flexural capacity of the slabs was exceeded in some locations.
- The SPR results indicated numerous locations where the cover to the top reinforcing steel was less than the specified value of 2 in.
- PB concluded the long-term durability of the SSTC was at risk due to tensile stress limits being exceeded and the low concrete cover.

Based upon the analysis results, PB recommended the installation of a 2 in. thick bonded latex modified concrete overlay on the slab surfaces. It is not clear if the overlay is intended as a structural overlay or as a bonded wearing surface. Cracks are expected to form in the overlay material.

The installation of the 2 in. overlay also has several possible complications, these include:

- The sidewalks and pedestrian areas drain through weep holes under the curb to the main drive areas. Installation of the two in. overlay would require extensive modifications (proposed details shown in the PB report dated May 22, 2012) to the drainage, with the resulting drainage "gutter" becoming a debris collection area and tripping hazard.
- Based upon the comments in the PB letter report dated May 22, 2012, the installation of the overlay will require extensive quality control testing to ensure adequate surface preparation and bond.

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2.3.5 Comments on STRUCTURAL Technologies Strengthening Plan

The pour strips on Level 330 were constructed without the design post-tensioning in the slabs. To address the deficiency, the installation of bonded fiber reinforced polymer (FRP) sheets and near-surface mounted rods are proposed. The proposed installation appears to address the strengthening requirements. However, we have several concerns regarding the proposed strengthening measures. These include:

- Verification that the proposed strengthening satisfies the limits presented in ACI 440 Section 9-2.
- The majority of the strengthening consists of FRP bars added to heavily cracked pour strips that the previous petrographic analysis indicated are susceptible to scaling. Need to verify that the existing concrete will be adequate to transfer the load and be durable.
- The quality control tests for the FRP materials (pull bond discs) are limited to sheet materials. Need to verify quality control procedures for the FRP rods.
- Need to indicate that surface penetrating radar or other means will be used to avoid damaging existing steel during FRP installation.
- Need a detail showing the proposed FRP bar installation including final top surface.
- Details showing CFRP sheet installation with finish coat were not provided.

Due to extensive cracking of the pour strip concrete, and concerns about the ability of the cracked concrete to transfer loads from the FRP bars into the concrete section, installation of the FRP for strengthening of the pour strips is not recommended.

3.0 FIELD INVESTIGATION

The field investigation by WDP included field observations and limited testing which included surface penetrating radar and impact-echo testing to assess the thickness of the concrete slabs. A series of probe openings were made to examine the condition of the grout used in the bonded post-tensioned slab construction. Resistivity tests were also performed on the concrete slabs surfaces to determine if significant variations in surface condition were apparent. Due to the extensive slab thickness testing performed by other parties, a detailed review of the slab thickness variation was not performed.

3.1 SITE OBSERVATIONS

Representatives of WDP made a series of site visits to the SSTC to examine the condition of the post-tensioned beam and slab members and the reinforced concrete columns. The following sections describe observations of specific conditions made during the investigation.

3.1.1 Slab Surfaces

The slab surfaces on the elevated levels are characterized by both random and regularly spaced cracks, with the most extensive cracks visible on the uncovered sections of Level 330 and on Level 350. The various crack surveys prepared by PB show an increase in the extent of cracking over time. Figures 3-1 to 3-7 show representative conditions observed at the SSTC during the field investigation.

The most extensive slab cracking was observed in the pour strips on Level 330 (Figures 3-3 and 3-4). The cracking in the pours strips appears to be the result of concrete shrinkage in both the longitudinal and transverse direction. The design post-tensioning as indicated in the PB drawings was not installed in the pour strip slabs, and consequently, the lack of precompression in the pour strips is likely contributing to the observed cracking.

Extensive cracking was also observed on the uncovered portion of Level 330. The cracking in this area was more extensive than on Level 350. There are several possible reasons for the extensive cracking in this area. These include:

- Casting of these areas was completed during the winter months where lower relative humidity levels likely contributed to plastic shrinkage cracking.
- These areas were cast nearly a year earlier than Level 350, which provided additional times for crack to occur.
- These areas were likely more extensively loaded and received more construction traffic which likely increased the extent of visible cracking.

In addition to the slab cracking, at approximately 10 locations, the bonded post-tensioning ducts were observed at the surface of the slab, and not covered with concrete. Exposed post-tensioning tendon ducts were not observed on the bottom surface of the slabs. The exposed post-tensioning ducts will need to be repaired to prevent damage to the post-tensioning tendons in service.

On Level 350, the concrete sidewalk and curb areas were placed on top of the structural deck. Water that penetrates through the sidewalk then drains through regularly spaced weeps at the curb line, onto the structural deck as shown in Figure 3-7. Drainage of water through the curb line will significantly influence any repairs to the concrete surfaces as the drainage path must be maintained. Further, the small weeps as detailed (Figure 3-8) are likely to clog in the future, acting to trap water beneath the sidewalk areas. Water trapped under the curb areas has the potential to cause damage under freezing conditions. These sidewalks will likely need to be replaced in the future.

In several areas, attempts have been made to repair the slab surface cracks (see Figure 3-1) by covering the cracks with epoxy and sand (Figure 3-9) and injecting the cracks with epoxy (Figure 3-10). These repairs have not been fully effective as the cracks have continued to grow in size and leakage through the cracks has continued to occur (Figure 3-11).

The surface crack repairs have been completed by filling (flooding) the cracks with epoxy and covering the crack with sand. These surface repairs have been ineffective due to the inability of the repaired cracks to accommodate thermal movement of the structure. Further, the cracks have extended over time due to long-term shrinkage of the concrete. To successfully repair the cracks, any material used to fill the cracks must have sufficient tensile strain capacity to accommodate movement and the repairs should be completed after the vast majority of shrinkage has occurred.

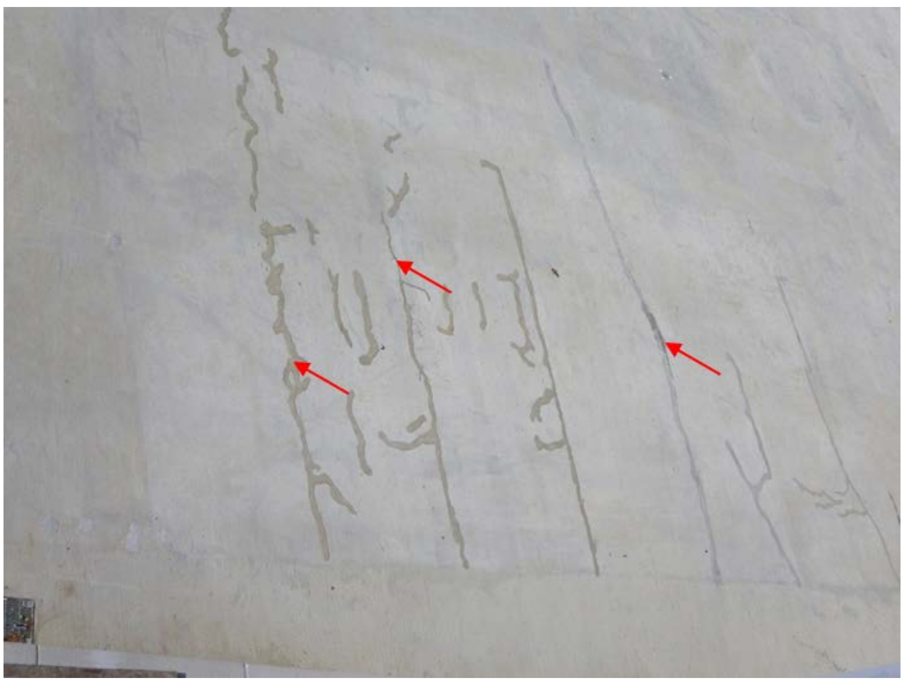


Figure 3-1. Map cracking on surface of Level 330 (note repaired cracks).

Figure 3-2. Map cracking on surface of Level 350.

Figure 3-3. Soffit of pour strip on Level 330.

Figure 3-4. Surface of pour strip on Level 330.

Figure 3-5. Exposed post-tensioning duct on Level 330.

Figure 3-6. Exposed post-tensioning duct on Level 350.

Figure 3-7. Curb condition in Level 330 with water evident at small diameter weeps.

Figure 3-8. Small weep hole located beneath curb area.

Figure 3-9. Crack on Level 330 covered with epoxy and sand.

Figure 3-10. Epoxy injected crack on Level 350.

Figure 3-11. Leakage (darkened areas) at injected cracks in pour strip on Level 330.

3.1.2 Beams and Columns

The beams and columns at the SSTC were visually surveyed to determine the extent of cracking present. Cracks were observed in both beams and columns. The cracking in the columns, shown in Figure 3-12 to 3-15, appeared to be the result of post-tensioning forces which acted to shorten the girders, resulting in the displacement of the columns. The crack widths were observed to vary from near hairline to approximately 0.030 in. The sandblasted finish of the column bases acted to make the cracks more visible and it harder to estimate the width of the cracks.

Figure 3-12. Cracking (highlighted) visible on Column B-6.

Figure 3-13. Approximate crack width of 0.016 in. on Column B-6 (above).

Figure 3-14. Cracking (highlighted) near base of Column C-1.

Figure 3-15. Approximate crack width of 0.030 in. near form seam on Column C-1 (above).

Cracks were also visible on the post-tensioned beams and girders in the SSTC, with more extensive cracking observed in the northeast portion of Level 330. The cracking in the beams and girders were likely the result of restrained shrinkage of the concrete prior to the post-tensioning forces being applied. In the northeast portion of the structure, additional restraint to concrete shrinkage was provided by the foundation wall on the north side of the structure. In some areas, evidence of water penetration through the cracks was observed. Representative cracking in the SSTC beams and girders is shown in Figures 3-16 and 3-17. The widths of the cracks in the beams and columns was typically 0.010 in. in width and smaller.

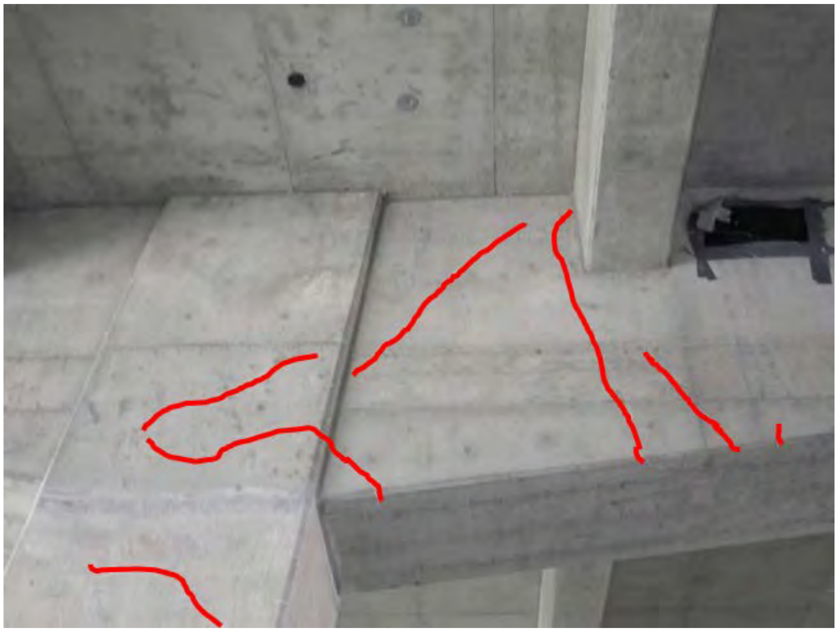


Figure 3-16. Cracks (highlighted) at beam-column joint on Level 330.

Figure 3-17. Cracking (highlighted) at beam-column joint.

Figure 3-18. Crack width measurement (0.008 in.) on beam face.

During the site investigation by WDP, numerous probe openings that had been made by other parties were observed to document the concrete cover over the reinforcing steel. Figures 3-19 and 3-20 show an example of low concrete cover over uncoated reinforcing steel in Column B-5. The measured cover at this location was 5/8 in., compared to the 1.5 in. required by ACI 318.

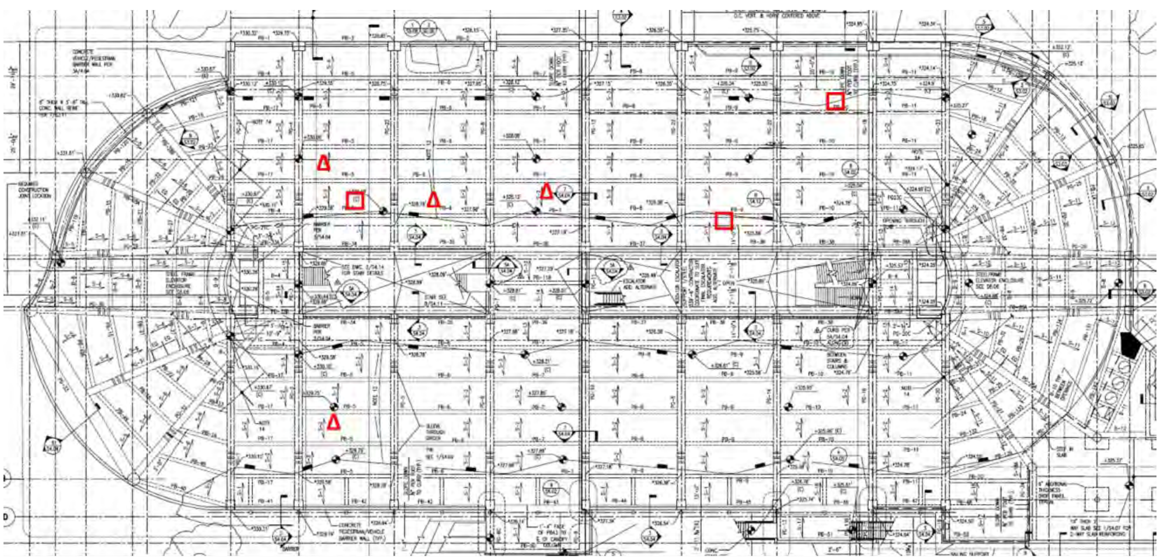
Figure 3-19. Column B-5 with low cover to reinforcing steel.

Figure 3-20. 5/8 in. cover to reinforcing steel.

3.2 INVASIVE PROBE OPENINGS

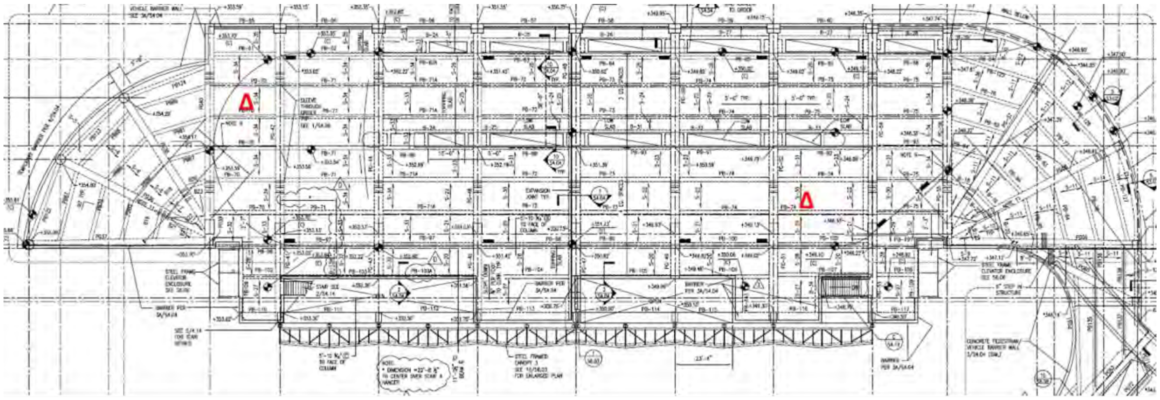
A bonded post-tensioning system was used in the SSTC construction. Due to concerns about the grouting of the post-tensioning tendons and the possible use of a grout material containing elevated levels of chloride that has been reported in the industry, a series of nine probe openings to examine the slab post-tensioning were completed. Figures 3-21 and 3-22 show the locations of the probe openings on Levels 330 and Level 350 respectively. The post-tensioning grouting records were reviewed to verify that the suspect grout material was used at the probe locations.

At each probe location, the slab post-tensioning tendon duct and reinforcing steel were located using surface penetrating radar prior to excavating the opening. The openings were made by removing the concrete using a combination of an electric chipping hammer and a cutting blade. The plastic duct was then removed by hand to expose the strand. At each location, a sample of the grout was taken for chloride testing. Extreme care was taken during the probes to ensure the prestressing strands were not damaged during the investigation. The probe openings were repaired by a concrete repair contractor after completion of the investigation.



Level 330 – Post-tensioning probe locations
Δ – soffit opening
□ - slab opening

Figure 3-21. Location of Level 330 probe openings.



Level 350 – Post-tensioning probe locations
Δ – soffit opening
□ - slab opening

Figure 3-22. Location of Level 350 probe openings.

3.2.1 Observations at Probe Openings

A total of nine probe locations were made to examine the condition of the grout in the slab post-tensioning. The nine openings included six opening on the slab soffits and three on the slab surface. The tendon ducts on the soffit were observed to be located within ACI 117 tolerances for cover concrete. The slab probes were located away from the locations where the post-tensioning ducts were observed at the slab surface (Figures 3-5 and 3-6).

At each location, the post-tensioning ducts were observed to be fully grouted with no evidence of grout subsidence or voiding. The prestressing strands in the ducts were observed to be clean with no evidence of corrosion products. Figures 3-23 to 3-25 show some of the conditions at the probe opening locations.

Figure 3-23. Fully grouted tendon duct on slab surface.

Figure 3-24. Well encapsulated prestressing steel in duct on slab soffit.

Figure 3-25. Prestressing steel with no visible corrosion products.

3.2.2 Chloride Test Results

Samples of grout were removed from each of the nine probe openings and tested for acid soluble chloride content. The chloride testing was performed in accordance with ASTM C1152-04[1] by a sub-consultant to WDP. The results of the chloride tests are shown in Table 3-1. The values in Table 3-1 are based on weight of the grout sample. However, the limits prescribed in ACI 318 are based upon weight of cement. Although the amount of cement in the grout samples is not specifically known, the measured chloride ion concentrations are less than the limit based upon weight of cement. Therefore, by mixture proportioning, are less than the limiting concentration based on weight of cement. All of the test results showed chloride contents less than the allowable value of 0.08 % by weight of cement that is contained in ACI 318.

[1] ASTM C1152-04 Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete, Published by ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA

Table 3-1 - Grout Chloride Sample Test Results

Level	Approx. Location	Chloride Content % by Wt.
330 Soffit	B6 – B5	< 0.010%
330 Soffit	B5 – B4	< 0.010%
330 Soffit	B4 – B3	< 0.0089
330 Soffit	C4 – C3	0.033%
350 Soffit	B3 – B2	0.042%
330 Slab	B4 – B3	< 0.0082%
350 Soffit	B8 – B9	0.011 %
330 Slab	B8 – B7	0.060%
330 Slab	B8 – B9	0.043%

3.3 NONDESTRUCTIVE TESTING RESULTS

A series of nondestructive tests were performed by WDP during the field evaluation. The nondestructive testing by WDP included surface penetrating radar (SPR) to measure concrete slab thickness and concrete cover, impact-echo testing to confirm SPR results of thickness and identify if there are internal flaws such as delaminations in the slabs, and concrete resistivity measurements to assess the relative resistance of the concrete to corrosion damage. Given the extensive nondestructive testing performed by other parties, the testing by WDP was intended to confirm previous results and was not intended to provide a comprehensive survey of the slab thickness.

3.3.1 Surface Penetrating Radar

Surface Penetrating Radar (SPR) is a nondestructive evaluation technique which utilizes electromagnetic energy to locate objects, subsurface flaws, or interfaces within a material. SPR was used during the SSTC investigation to examine the thickness of the slabs and to verify the location of embedded reinforcement.

WDP

SPR utilizes a high frequency dipole antenna to transmit a train of discrete amplitude modulation (AM) radio wave pulses. A second antenna, housed next to the transmitting antenna, is used to receive the scattered pulses as they return to the surface of the material. The radar unit detects back-scattered radiation that is reflected at the boundary between differing dielectric media. By measuring the time it takes to receive the reflected signal, the depth of an embedded object or interface may be determined. A real-time visual display of the material cross section is recorded as the antennae are moved along the surface. The output is fed to sampling circuitry before being digitally processed by a computer.

The color and intensity of the patterns in the output are related to the amplitude of the reflected signals. Figure 3-26 illustrates SPR sample scanning and signal output. The bands of alternating light and dark areas which appear in the remainder of the output correspond to the positive and negative reflections of the input wave from subsurface objects. Reflections from a lower dielectric media to a higher one, such as from concrete to steel, will undergo a phase inversion, or reversal, and the boundary will show up as a bright signal (assuming the input wave is dark). Conversely, the boundary from concrete to air (higher dielectric to lower) will show up as a dark signal. SPR antennae are specifically tuned to detect cylindrical objects, such as reinforcement, conduit, pipes, etc. These types of features show up as hyperbolas, or arch shapes, on the data record, as seen in Figure 3-26.

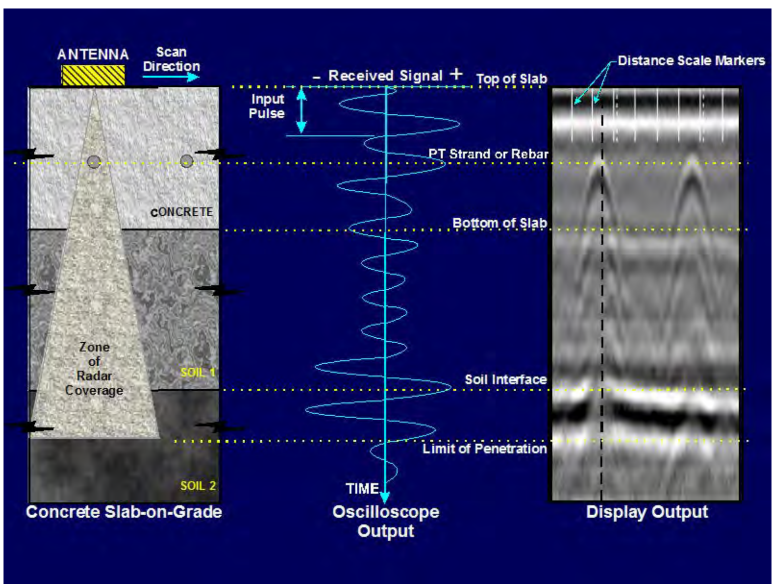


Figure 3-26. Sample output from a SPR scan.

SPR was used to scan sections on Levels 330 and 350 where PB determined the concrete slab thickness was less than the 10 in. design value. The SPR results by WDP confirmed this finding. Figure 3-27 shows representative SPR scan from Level 330. In the figure, the bottom of the slab can be clearly seen. SPR was also used in the investigation to locate reinforcing steel and post-tensioning ducts at the probe openings.

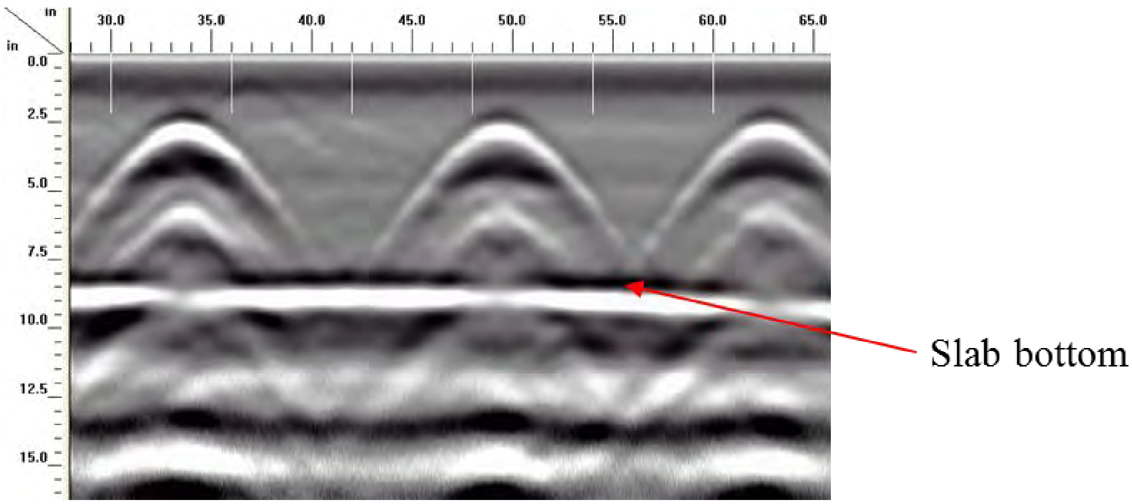


Figure 3-27. SPR test result from Level 330. Slab thickness approximately 8 in.

3.3.2 Impact-Echo

Impact-echo testing is a nondestructive stress-wave based test method for concrete and masonry structures. The method was used at the SSTC to examine the thickness of the concrete slabs on Level 330. An impact-echo test system is comprised of three components: an impact source, a receiving displacement transducer, and a portable computer with a data acquisition card. The impact source is a hardened steel sphere on a spring steel rod. Different size impactors are utilized during the testing in order to vary the frequency content of the stress waves generated by the impact. Higher frequency (shorter wavelength) components are needed for testing thinner structures or detecting small or shallow flaws. The receiving transducer is a broadband piezoelectric displacement transducer; a thin lead strip is used to provide acoustic coupling between the transducer and the concrete surface. A portable computer, complete with data-acquisition card and software, is used for signal acquisition and analysis.

When conducting an impact-echo test, a transient stress pulse is introduced into the member by striking the impactor on the surface as illustrated in Figure 3-28. The stress pulse propagates into the object along spherical wavefronts. The waves are reflected by internal cracks or interfaces, such as voids, cracks, and delaminations, as well as by the external boundaries of the member. The arrival of these reflected waves at the surface where the impact was generated produces surface displacements that are monitored by the transducer. If the transducer is placed close to the impact point, the waveform is dominated by displacements caused by P-wave arrivals.

The arrival of reflected P-waves over time creates a time-domain waveform (voltage versus time). Because it can be difficult and time-consuming to analyze the time-domain waveform, the signal is typically transformed into the frequency domain (amplitude versus frequency) by the fast Fourier Transform (FFT) technique. Multiple P-wave arrivals from an interface, such as a void, delamination, or an external boundary, will be revealed by frequency peaks in the amplitude spectrum. If the wavespeed in the concrete, C_p, is known, the depth to the reflecting interface can be calculated with the following equation:

$$T = 0.96 \frac{C_p}{2 f_p}$$

In the equation, T is the depth to the interface and f_p is the frequency of the P-wave reflections from the interface. The value of 0.96 is a factor that corrects for flexural vibrations of the slab or wall during testing. The wavespeed in the concrete can be measured from a surface wave or from an element of known thickness. In slabs and walls, wave reflections from the side boundaries do not have a significant effect on the response. At the SSTC, the wavespeed in the concrete was verified at core locations where a known thickness was preset.

The impact-echo tests were conducted on Level 330 between column lines C5 and C4. Testing by PB revealed a concrete slab varying from 10.5 to 8 inches in this area. A wavespeed of 4,000 m/sec. was used in the testing, with the value verified at core hole. Figures 3-29 to 3-31 show representative impact-echo test results indicating a concrete slab thickness of approximately 10” (256 mm), 9” (228 mm) and 8” (205 mm). These results are consistent with the results presented by PB.

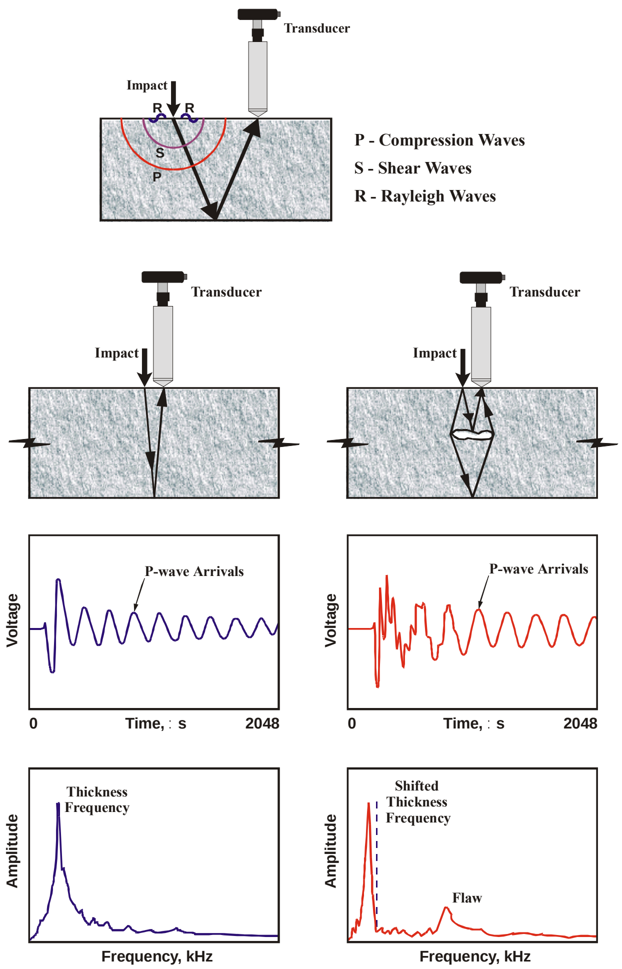


Figure 3-28. Schematic of the impact-echo method.

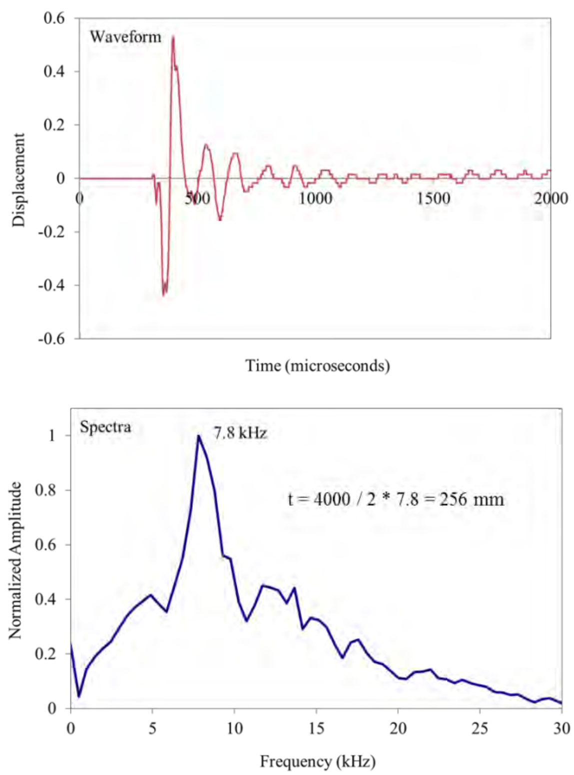


Figure 3-29. Impact-echo test result showing an approximate 10” slab thickness.

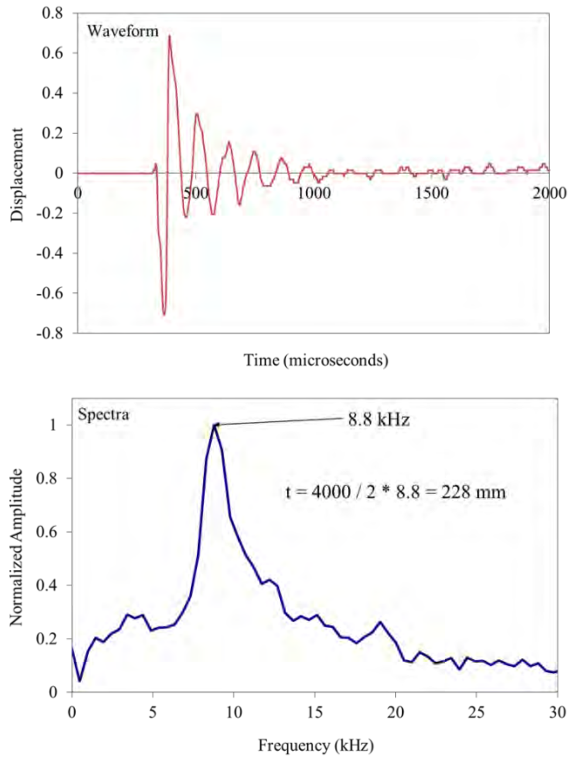


Figure 3-30. Impact-echo test result showing an approximate 9" slab thickness.

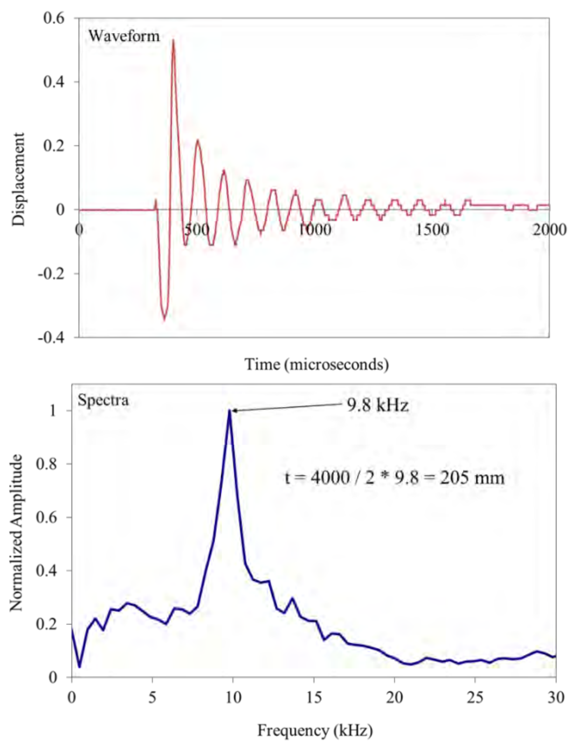


Figure 3-31. Impact-echo test result showing an approximate 8" slab thickness.

3.3.3 Concrete Resistivity

To assess the expected durability of concrete slabs, a series of resistivity tests were performed. The resistivity tests (AASHTO TP 95-11[2]) are intended to provide a measure of the ability of the concrete to resist the penetration of chloride ions, which act to accelerate reinforcing steel corrosion. Concrete with resistance will inhibit the flow of electrons between anodic and cathodic sites and thus limit the potential for corrosion of embedded steel. The electrical resistance of concrete is dependent upon the microstructure of the paste and the moisture content of the concrete. The technique requires four equally spaced electrodes as shown in Figure 3-32 below (s = 1.5 in. for WDP device to be consistent with the AASHTO Standard). The test method is shown graphically below. The two outer electrodes are connected to an alternating current source while the inner two electrodes measure the resulting voltage generated between the inner electrodes. The resistivity of the concrete is determined by dividing the measured voltage by the current applied with an adjustment to account for the spacing of the probes.

WDP

[2] Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration, Published by American Association of State and Highway Transportation Officials.

Resistivity test results are categorized on a log scale to assess the relative resistance of the concrete to chloride ion penetration. Table 3-2 shows the relationship between concrete resistivity and permeability used in the assessment of the SSTC concrete.

Table 3-2 - Permeability of Concrete based upon Surface Resistivity[2]

Permeability	Surface Resistivity (kΩ-cm)[1]
High	< 10
Moderate	10 – 16
Low	17 – 29
Very Low	29 – 199
Negligible	> 199

1. Based upon a 6 in. by 12 in. cylinder.

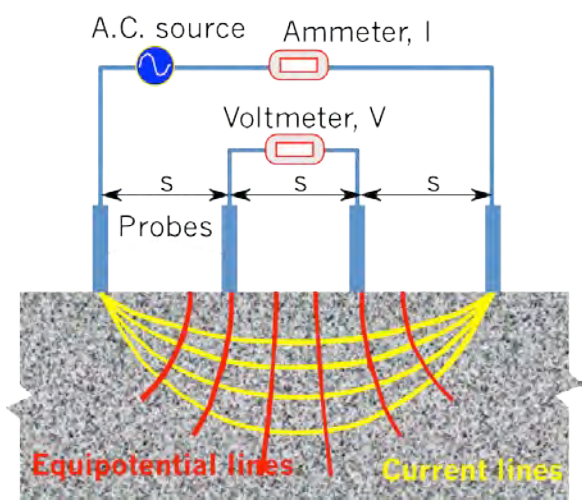


Figure 3-32. Schematic of resistivity test method.

Resistivity tests were performed at 7 locations at the SSTC. WDP was unable to perform resistivity tests on the Level 330 west pour strip due to a coating that was present on the surface in this area. At each resistivity location area, a 1 ft. by 1 ft. grid was set up with tests performed at each grid point, with a sufficient grid size to obtain a minimum 24 test results. Figure 3-33 shows a resistivity test in progress. The results of the resistivity testing are summarized in Table 3-3 with the testing locations shown in Figures 3-34 and 3-35.

WDP

Figure 3-33. Resistivity test being performed.

Table 3-3 – Summary of Surface Resistivity Test Results

Area	# tests	High (kΩ-cm)	Low (kΩ-cm)	Mean (kΩ-cm)
Level 330 C8 - C9 Midspan	25	227	106	160
Level 330 B5 - B4 Midspan	36	442	176	220
Level 330 C5 - C4 Midspan	24	289	154	240
Level 330 B8 - B 7 Midspan	30	332	172	220
Level 350 A6 - A7 Midspan	30	207	92	150
Level 350 A5 - A4 Midspan	36	200	120	150
Level 330 – East Pour strip	30	246	78	140

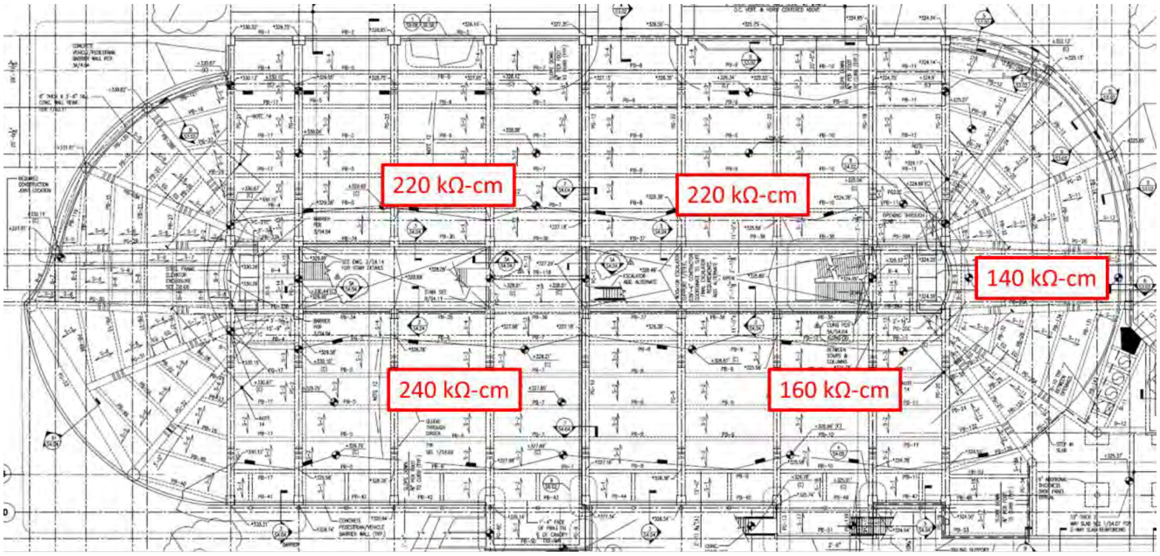


Figure 3-34. Level 330 resistivity test locations with mean resistivity value shown.

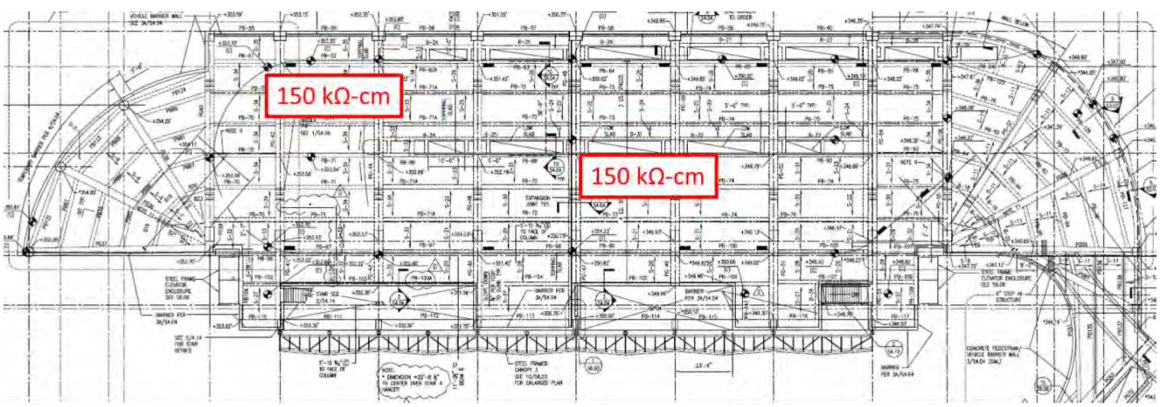


Figure 3-35. Level 350 resistivity test locations with mean resistivity values shown.

The resistivity testing indicated the concrete had a relatively high level of resistivity and would be considered resistant to chloride ion penetration in areas away from cracking. The areas with the lowest measured resistivity were the areas with more extensive surface cracking. Concrete resistivity is inherently a property of the concrete, and the measured resistivity values represent the resistivity of the material in the vicinity of the test location. The lower resistivity values in the areas with more extensive surface cracking is likely related to the higher moisture content (water in cracks) in these areas compared to areas with fewer cracks.

3.4 DISCUSSION OF WDP FIELD INVESTIGATION RESULTS

The field investigation by WDP was intended to visually survey the existing conditions in the structure, to verify the results obtained in previous investigations by other parties, to examine the condition of the grout in the bonded post-tensioning and to examine the general quality of the concrete construction. The visual survey indicated that extensive cracking was present on the elevated slab surfaces, and the length of cracks had increased compared to previous surveys by PB.

The impact-echo and SPR testing by WDP confirmed that sections of the post-tensioned slabs were thinner than required by the SSTC design requirements. The invasive probes indicated that the grout used for the bonded post-tensioned slabs did not contain chlorides and that the examined sections were well grouted. Resistivity testing on the concrete slabs verified that the concrete in the elevated slabs was resistant to chloride penetration in uncracked concrete. The presence of surface cracking will allow for greater amounts of chloride ingress to the reinforcing steel.

Long-term protection of the reinforcing steel from corrosion will require both adequate concrete cover and high concrete resistivity. Based upon the extensive cracking on the elevated slabs and the petrographic analysis results, a long-term protection strategy will be needed for the elevated slabs to achieve the prescribed 50 year design service life.

4.0 STRUCTURAL ANALYSIS

To evaluate the structural design of the SSTC, an analytical model of the structure was developed. The goal of the structural analysis was to determine if the SSTC, both as originally designed and as-built satisfy the WMATA design criteria. In the analysis, representative sections were examined at service and factored load levels to assess the stress levels and member capacity. Additional analyses were performed to examine the impact of the thin slab sections.

The model developed by WDP was a three-dimensional model of the entire structure that examined the behavior of the structure during the construction phase, at service conditions and at ultimate strength. The analysis was linear elastic in nature, therefore, redistribution of moments and other non-linear analytical methods were not considered. The following sections describe the results of the structural analysis completed by WDP.

4.1 MODEL DESCRIPTION

The analytical model for the SSTC was developed to allow for the adequacy of both the as-design and as-constructed structure to be examined. The geometry of the three-dimensional structure was developed based upon the ASI construction set drawings and VSL shop drawings. The post-tensioning forces measured during construction were used in the analyses. The following sections describe the analytical models used by WDP.

4.1.1 Analysis Model

The analysis model was developed using SAP, a general purpose structural analysis program. All of the analyses were linear elastic. The model was developed using a combination of shell elements for the slabs, frame elements for the beams and columns. Solid elements were added at the beam column joints to represent the additional concrete added in these areas. The centroid of the beam elements were offset from the centroid of the shell elements to accurately represent the stiffness of the beam / slab construction. The expansion joint along column line 6 was modeled using links to allow for horizontal displacement (thermal movement) of the slabs and transfer of vertical loads. Figure 4-1 shows the final model of the SSTC structure.

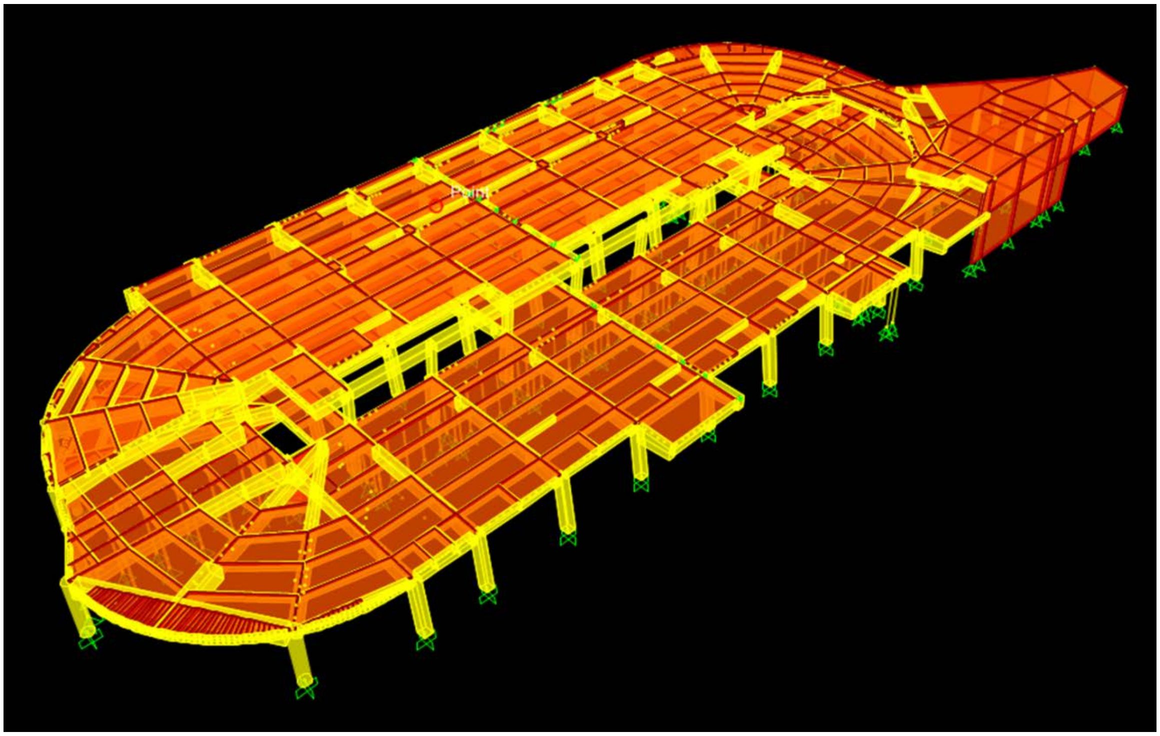


Figure 4-1. Completed model of SSTC structure.

The post-tensioning tendons were modeled as embedded elements within the shell and beam elements. The post-tensioning tendon profiles and initial stressing force (jacking force) were developed from the approved post-construction post-tensioning shop drawings.

The focus of the analysis was on the capacity of the post-tensioned slabs, beams and girders exposed to vehicular loading. Therefore, some components of the structure were not included in the model. These components included:

- Stairways between levels.
- Steel canopies around the perimeter of the SSTC.
- Escalators / elevators and associated framing members.
- East end "Green" roof.

4.1.2 Applied Loadings

The applied loadings on the model included applicable dead and live loads as specified in the design documents. Lateral loads arising from wind and seismic forces were not considered in the analysis. The loadings from wind and seismic forces will not control the design of the beam and slab elements, and were therefore not considered. The design loads examined by WDP included:

- Dead load of the structure.
- Superimposed dead load of 35 psf in drive lanes for future wearing surface.
- HS 25-44 Design vehicle in drive lanes with 33% impact factor.
- Live Load = 150 psf in pedestrian / sidewalk areas.
- Snow loads (on open areas) based upon P_g = 30 psf. Reduced to 25 psf on the exposed elevated slabs.

The HS-25 design vehicle used in the analysis was developed from the AASHTO Standard HS-20 design vehicle. Figure 4-2 shows design vehicle with HS-25 axle loads. Using the 33% impact factor, the point loads associated with the individual wheel loads were determined to be 26.6 kips for trailer wheels and 6.7 kips for the front wheels. Similar loads were used in the evaluation of the pour strips by Structural. The reduction factor used in AASHTO that accounts for the presence of multiple loaded vehicles was not used in the analysis.

The location and number of design vehicles in the structure were varied to produce maximum forces on the slabs, beams and girders. Based upon the typical beam and slab spans, a distance of 14 ft. was used between the trailer axles. Figures 4-3 to 4-9 show the design vehicle configurations used in the analysis.

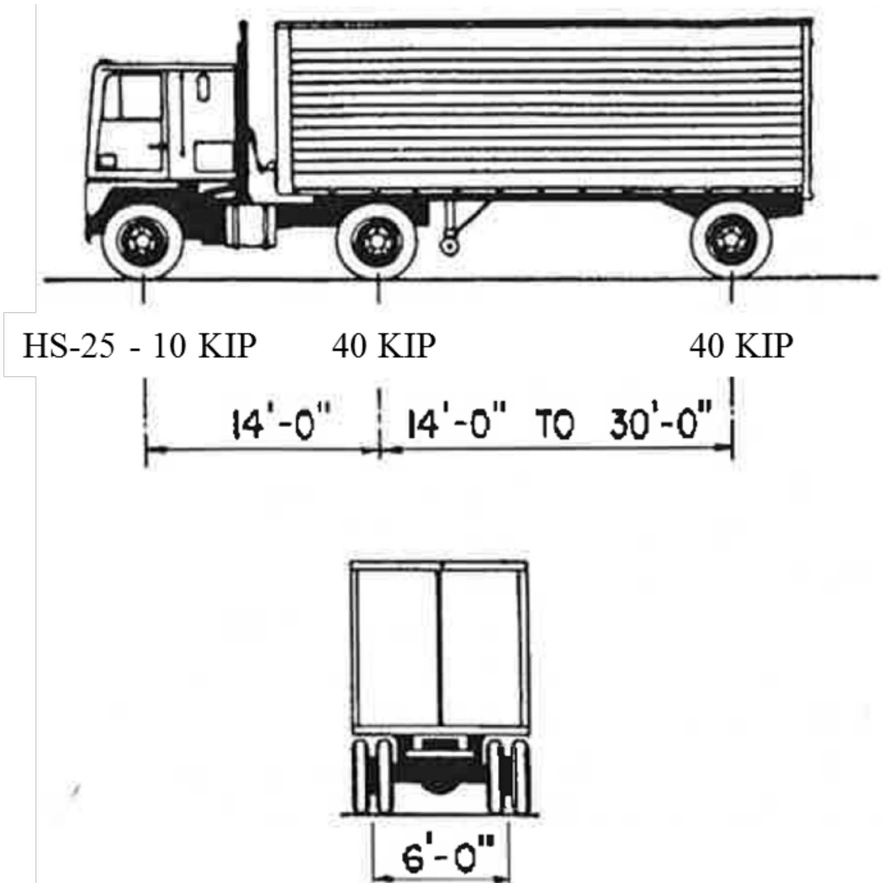
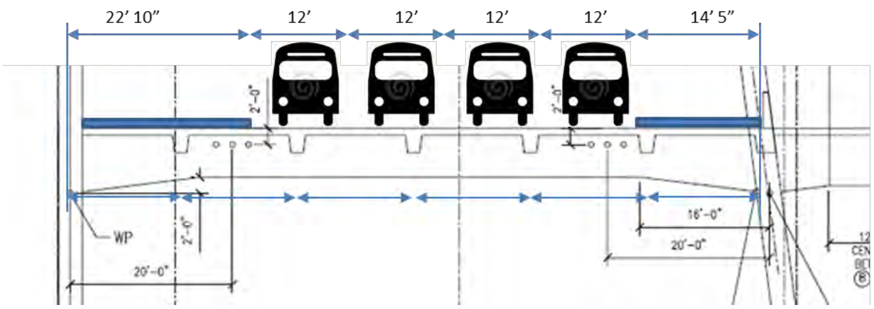


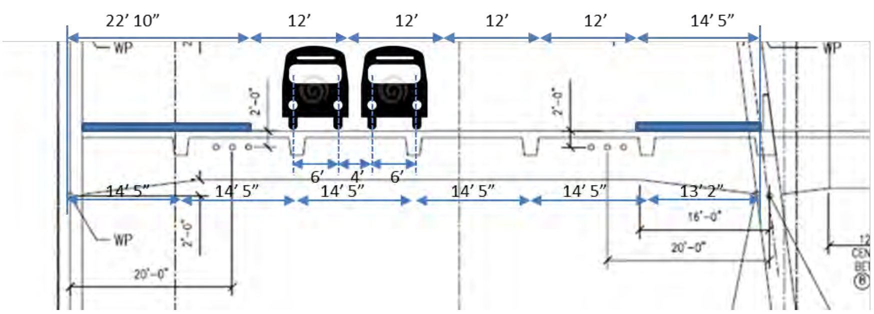
Figure 4-2. HS-25 design vehicle used in analysis model.

To simplify the analysis, the various loads were placed on Level 330 as shown in Figures 4-7 to 4-9. The loading location is shown in Figure 4-10. Note that previous evaluation by STRUCTURAL Technologies have indicated that the pour strip sections on Level 330 were inadequate as they did not contain post-tensioning tendons. Additional calculations to examine the strength of the improperly reinforced pour strips were not completed.



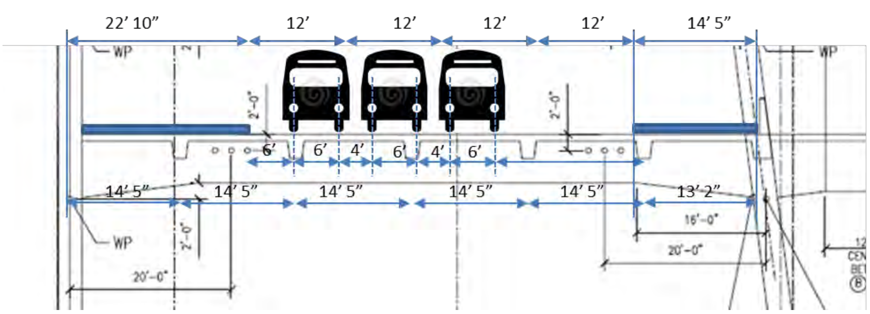
Load Case 1:
Typical load case

Figure 4-3. Deck analysis – typical load case.



Load Case 2:
Worst condition
for deck positive
moment

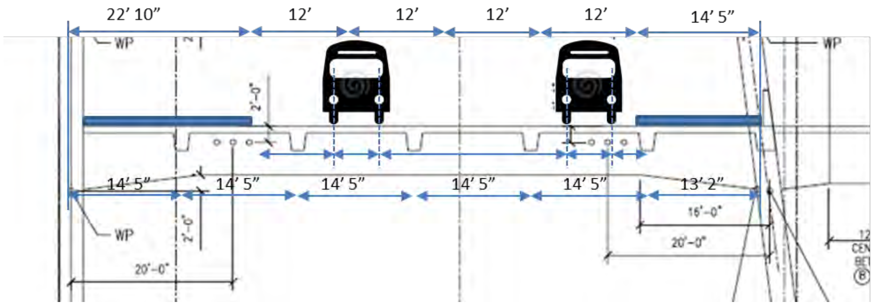
Figure 4-4. Deck analysis – load case for maximum positive moment.



Load Case 3:
Worst condition
for deck negative
moment

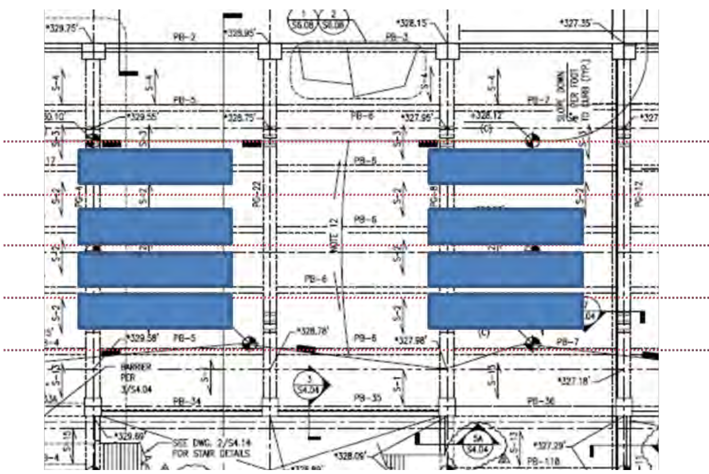
Figure 4-5. Deck analysis – load case for maximum negative moment.

WDP



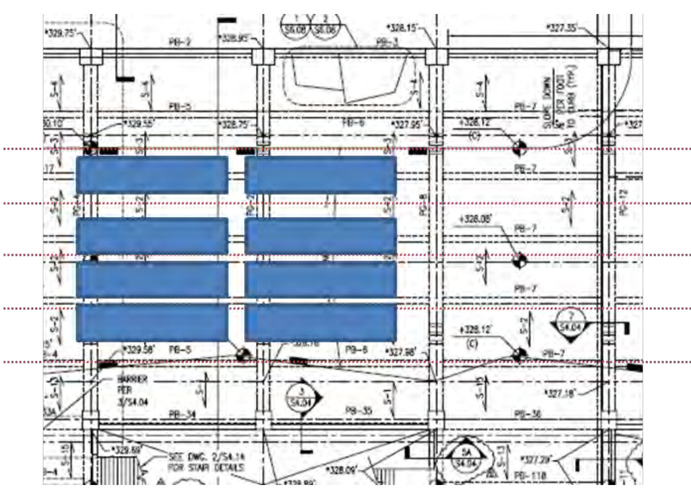
Load Case 4: Reverse bending at deck mid span

Figure 4-6. Deck analysis – load cast for reverse bending in slab.



Beam Load Case 1: Worst condition for beam positive moment and reverse bending at beam mid span

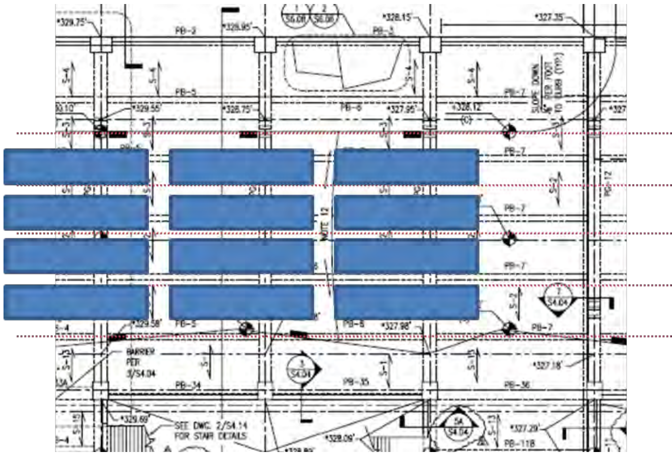
Figure 4-7. Beam analysis – load case for maximum positive moment and reverse bending.



Beam Load Case 2: Worst condition for beam negative moment

WDP

Figure 4-8. Beam analysis – load case for maximum negative moment.



Girder Load Case 1: Worst condition for girder positive and negative moments

Figure 4-9. Girder analysis – load case for maximum positive and negative moment.

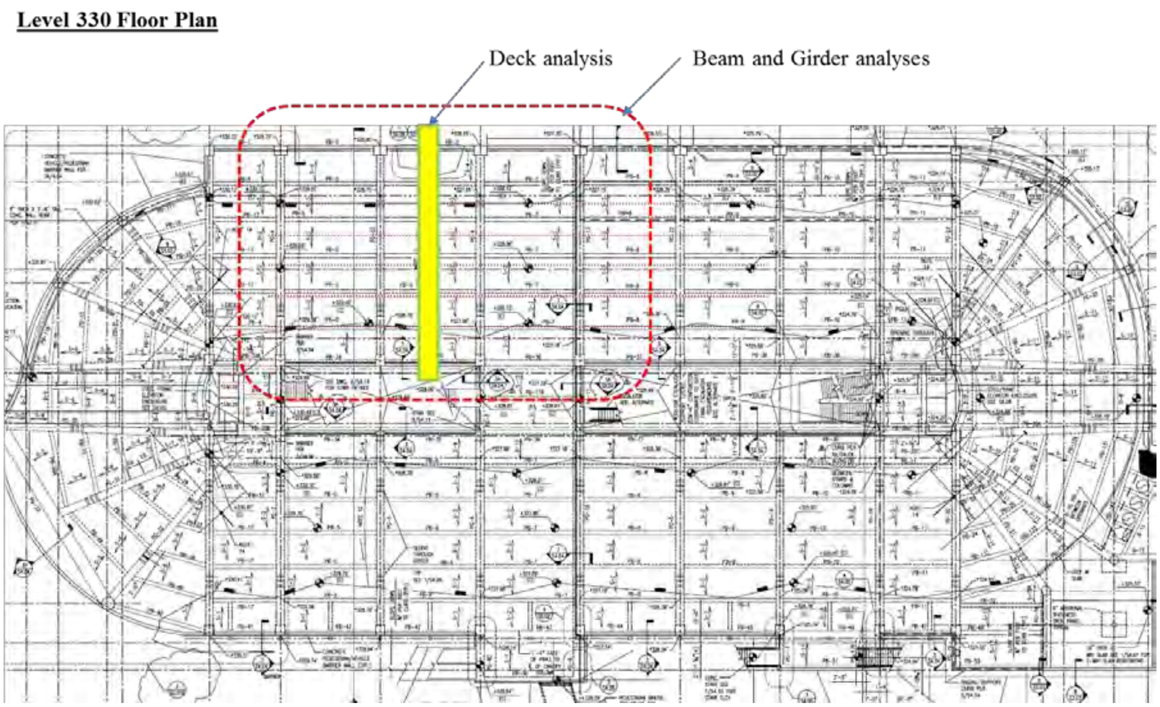


Figure 4-10. Representative section of Level 330 with loaded areas shown.

4.1.3 "Thin Slab" Evaluation

Testing by numerous parties determined that the thickness of the post-tensioned slabs were less than the 10 in. design thickness. To examine the effect of the thin slab, the SAP model of the structure was modified to reduce the slab thickness to 8 in. in the area shown in Figure 4-11. The post-tensioned beams and girders thicknesses were also reduced in the area. The reduced slab area is similar to the results of the laser scan performed by PB and confirmed in the field testing by WDP. The slab capacity in the thin slab area was determined based upon an assumption of a 1 in. decrease in cover.

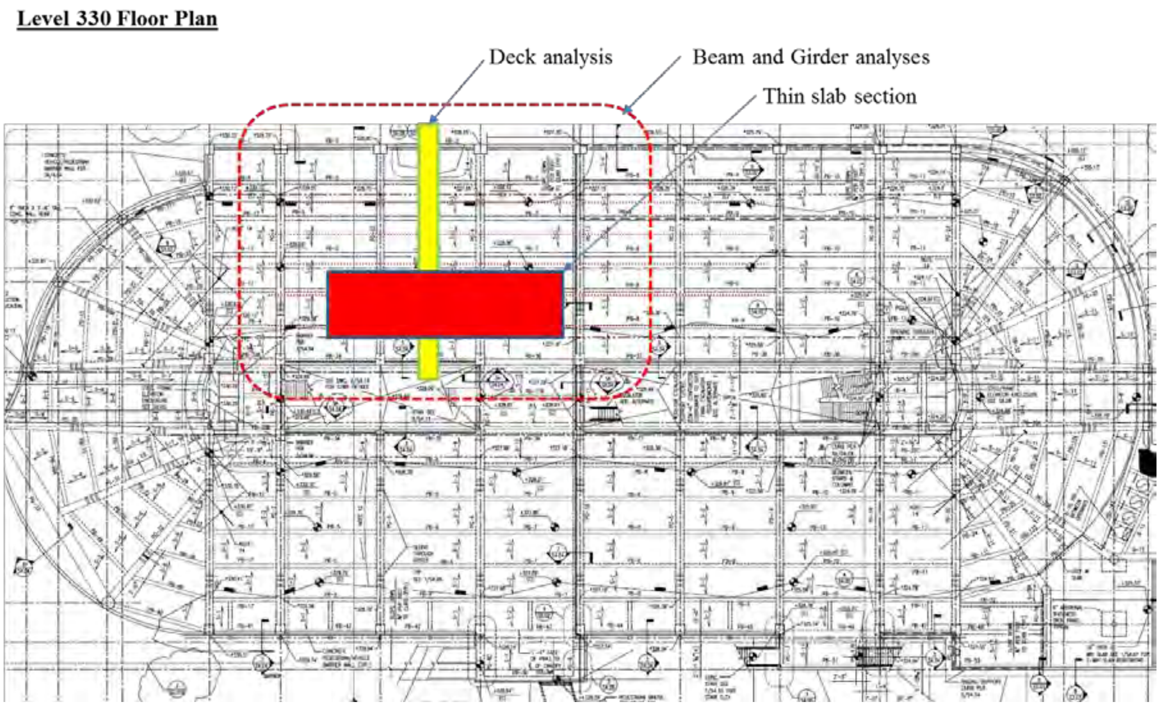


Figure 4-11. Representative section of Level 330 with thin area (8 in.) shown.

4.1.4 Material Properties

The materials properties used in the analysis were based upon the design values for the concrete, reinforcing and prestressing steel. These values are summarized in Table 4-1.

Table 4-1 – Material Properties used in SSTC Evaluation

Material	Young's Modulus (ksi)	Compressive / Yield Strength (ksi)	Ultimate Strength (ksi)
Concrete	5,100 ksi	8 ksi	-
Prestressing Strand	28,500 ksi	245 ksi	270 ksi
Reinforcing Steel	29,000 ksi	60 ksi	-

The concrete strength used in the analysis was based upon the design strength of 8,000 psi. A higher strength was used in the analysis by SGH, however this value was not supported by core strength test results and therefore was not used.

The allowable service load stress limits, used to determine if the structure satisfied the WMATA design criteria, were based upon the 8,000 psi compressive strength of the concrete. Transfer stress limits were based upon a compressive strength of 3,500 psi at transfer (f'_{ci}). The allowable stress limits are summarized in Table 4-2.

Table 4-2 – Allowable Stress Limits

Stress State	Limit[1]	Value (ksi)
Compressive Stress (transfer)	$0.6\, f'_{ci}$	2.10 ksi
Compressive Stress (service)	$0.6\, f'_c$	4.80 ksi
Tensile Stress (transfer)	$6 \sqrt{f'_{ci}}$	0.350 ksi
Tensile Stress (service)	$6 \sqrt{f'_c}$	0.540 ksi

1. f'_{ci} – compressive strength of the concrete at transfer.

In the analysis models, the post-tensioning forces in the slab, beam and girders were based upon the forces shown in the post-tensioning shop drawings. Losses in post-tensioning force due to concrete shrinkage and creep and steel relaxation were assumed to be 20%.

4.1.5 Load and Strength Reduction Factors and Load Combinations

The analysis of the SSTC was based upon the load and strength reduction factors and load combinations specified in ASCE 7 (referenced in the IBC code). Accordingly, load factors of 1.2 and 1.6 were used for dead and live loads, respectively. The governing load combination on Level 330 for strength was found to be the basic load combination shown below:

Design load = 1.2 DL + 1.6 (LL + Impact) + 1.0 Hyperstatic

Where hyperstatic refers to the secondary moments created by the reactions from post-tensioning forces. The strength reduction factors (from ACI 318) used in the analysis was 0.9 for flexure, and 0.75 for shear and compression. Note that the original SSTC design by PB, load factors of 1.4, 1.7 and 1.0 were used for deal, live and hyperstatic loads respectively.

4.2 CONSTRUCTION PHASE ANALYSIS

The SSTC was built in a phased manner, accordingly the post-tensioning forces on the structure varied as the different components of the structure were added. Table 4-2 shows the reported dates for the various components of the SSTC. To examine the stresses in the structure as construction progressed, a phased construction analysis was performed. The intent of the analysis was to examine if the restraint provided by different components (such as the north perimeter wall or the girders) affected the post-tensioning force in the various components of the structure. This analysis creates a more realistic look at the actual post-tensioning forces developed in the structure as restraints are more accurately represented.

Table 4-3 – Pour Dates for SSTC Construction

Pour Name	Date	Pour Area (SF)
1A	9/13/2010	11,030
1B	10/2/2010	12,000
1C	10/18/2010	12,700
2A	11/2/2010	6,400
1Ea	11/12/2010	6,300[1]
2B	12/7/2010	15,900
1Eb	12/10/2010	6,300
1D	12/20/2010	9,500
1F	12/30/2010	7,700
East Pour Strip	1/12/2011	760
2C	1/14/2011	11,700
2D	1/31/2011	8,400
2Ia	3/29/2011	4,500[2]
West Pour Strip	4/19/2011	760
2Ib	5/20/2011	4,000[2]

1. Total pour area – 12,600 SF – assumed to be split evenly
2. Total pour area – 8,500 SF – split into two areas of 4,000 and 4,500 SF

In the phased construction analysis, for each SSTC section, the structural model was created and then dead loads and post-tensioning forces were applied. In some locations, temporary supports were included in the model to support cantilever section. Temporary post-tensioning was also modeled based upon the VSL shop drawings.

Based upon discussions with WMATA personnel, it is our understanding that some of the post-tensioning stressing operations were performed using a single-strand jack in lieu of the expected multi-strand jack. The post-tensioning force in the phased construction model was applied in a single step, in lieu of attempting to discrete application of post-tensioning forces. Application of post-tensioning forces in single step does introduce slight differences in the post-tensioning force (due to differences in elastic shortening) compared to discrete application using a multi-strand jack. The difference is not considered to be significant.

Results from the construction phase analysis were compared to results from a "traditional" analysis to determine if the construction sequence has a significant impact on the stresses in the members at the time of post-tensioning transfer.

4.2.1 Construction Phase Results

The results of the construction phase analysis are shown in Figures 4-12 to 4-15. In each figure, a different stage of construction is represented.

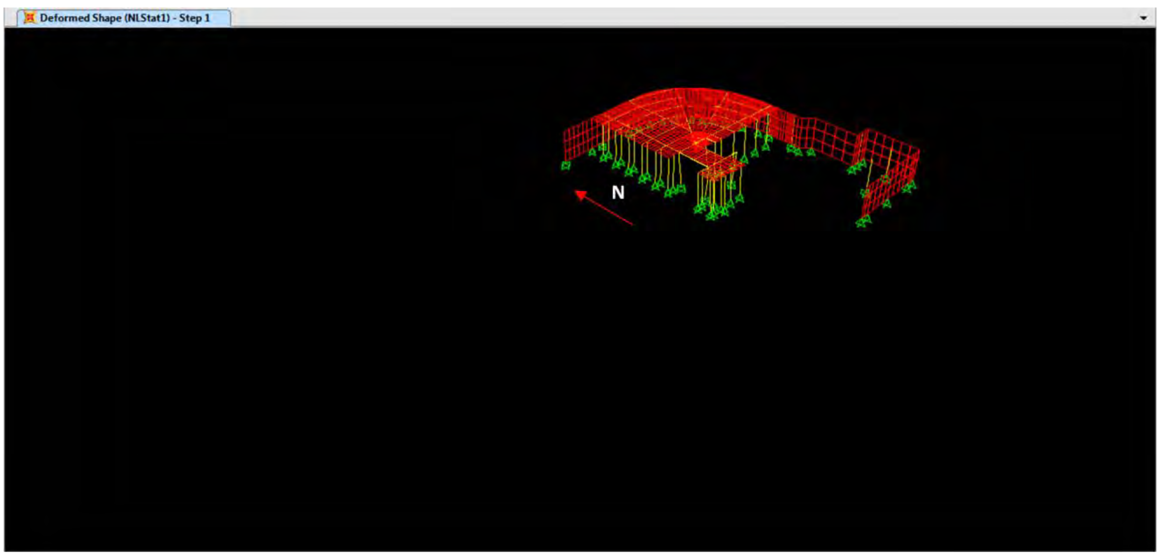


Figure 4-12. Initial construction phase – pour 1A.

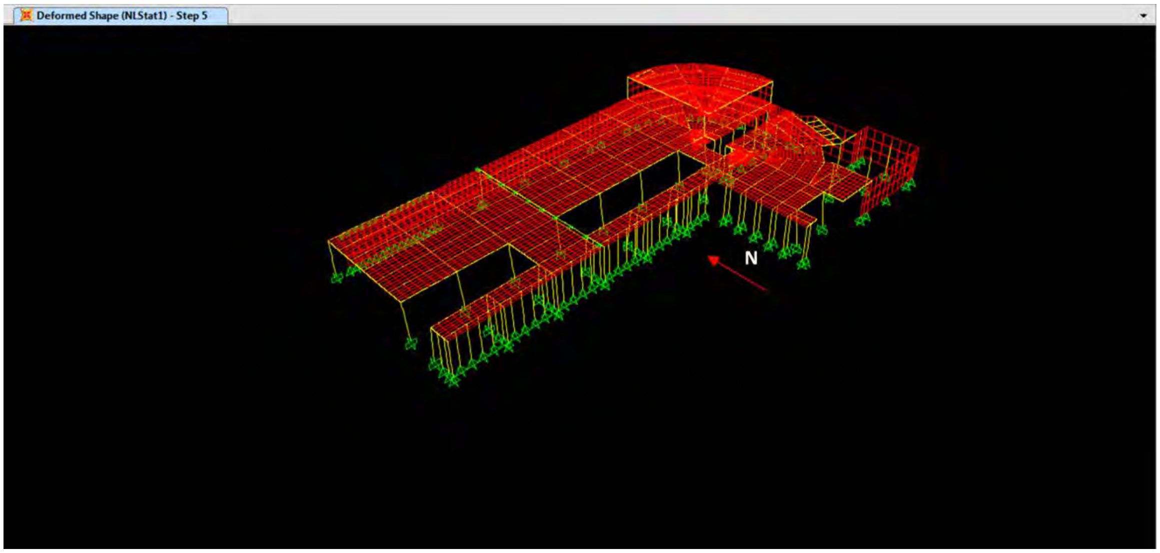


Figure 4-13. Completion of first stage on Level 350.

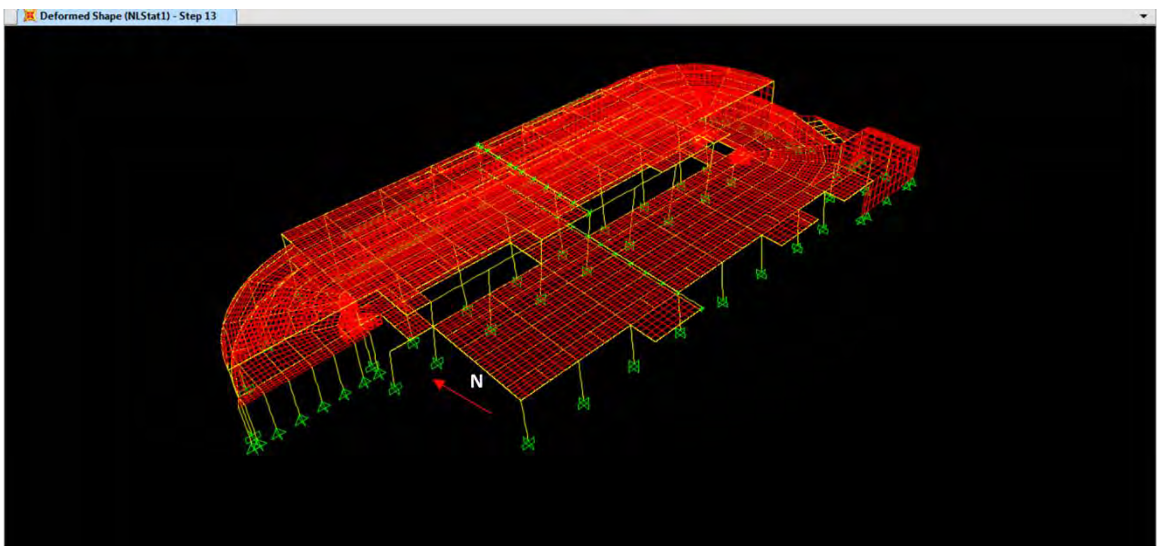


Figure 4-14. Construction after casting of east pour strip.

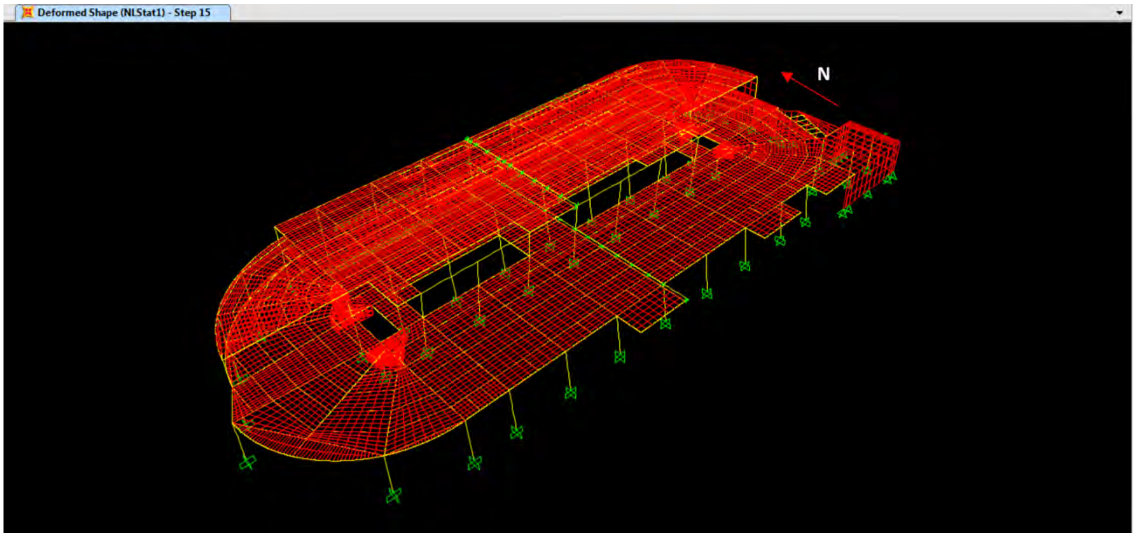


Figure 4-15. Final construction phase.

The results of the construction phase analysis indicated that the stresses in the structure at the time of transfer were within allowable stress limits. Representative stress contours from the analysis results are shown in Figures 4-16 to 4-19. A portion of the structure, in SE corner, was constructed using a two-way slab without post-tensioning and therefore the stress contours in this area are not critical.

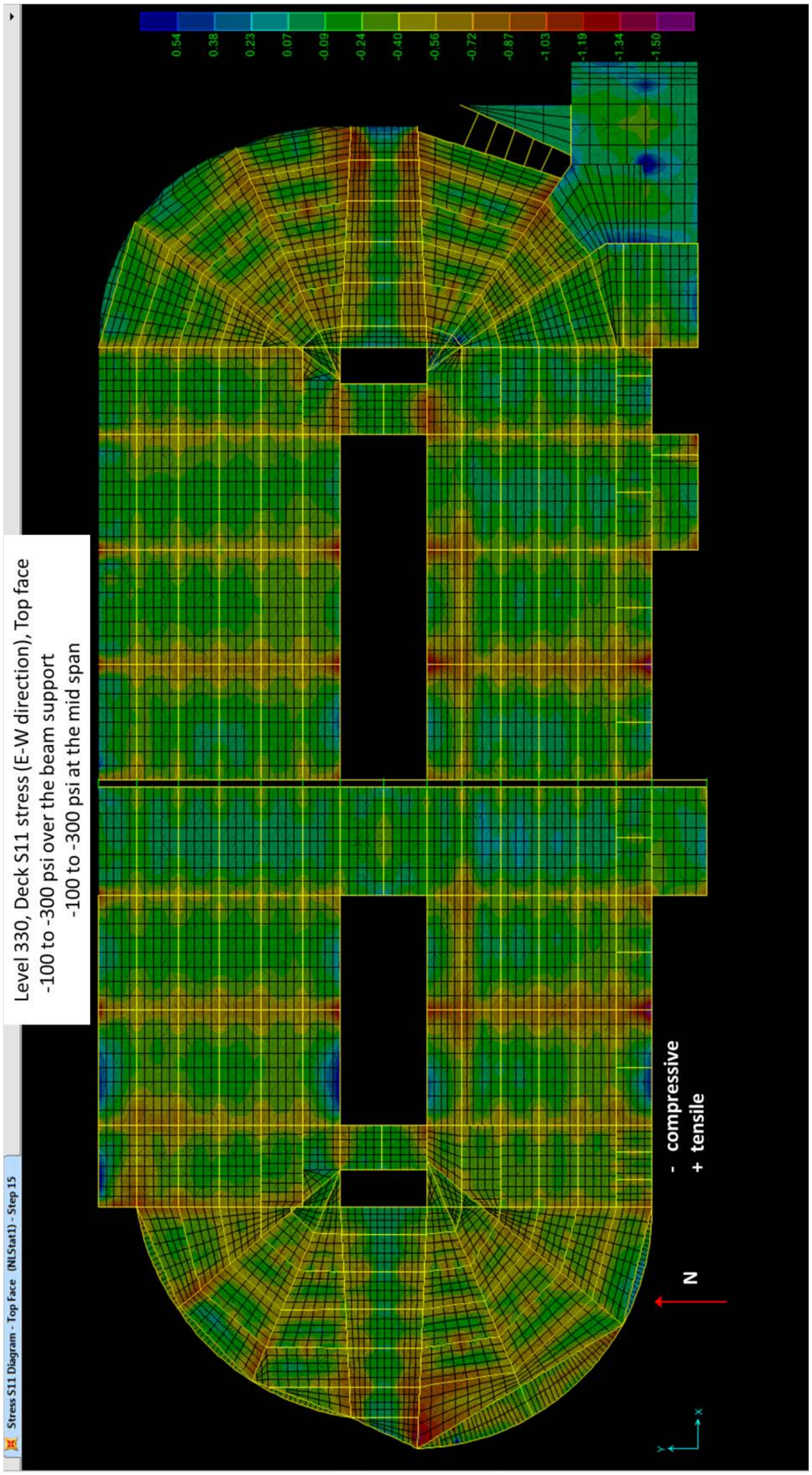


Figure 4-16. Stress contour on Level 330 after construction.

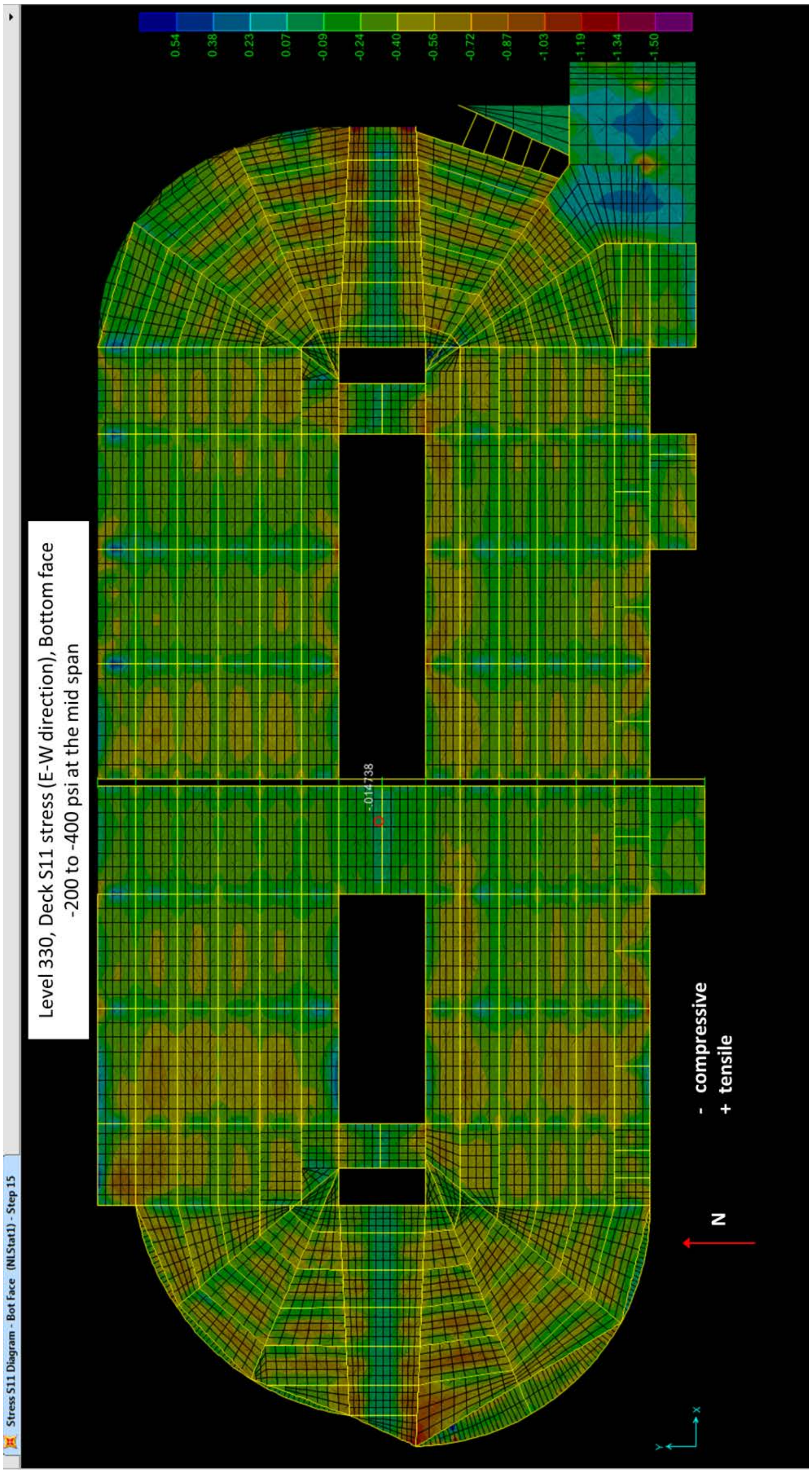


Figure 4-17. Stress contour on Level 330 after construction.

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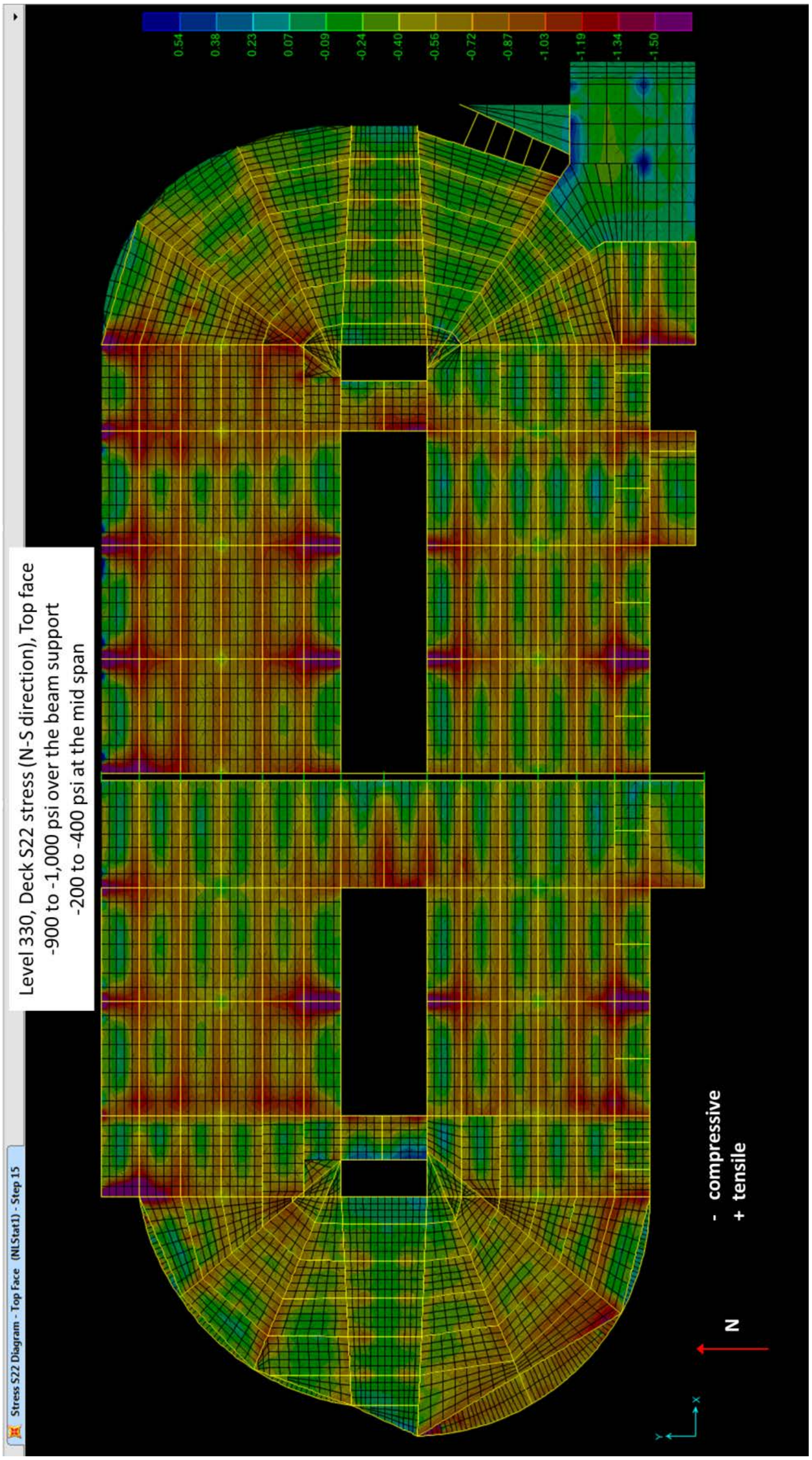

Figure 4-18. Stress contour on Level 330 after construction.

Figure 4-19. Stress contour on Level 330 after construction.

The results of the construction phase analysis indicated that the slabs, beams and girders of the SSTC were in compression after completion of the stressing operations. Isolated locations with high tensile stresses were observed in the analysis results, these were attributed to idealization of construction support conditions in the models.

To examine the difference between the construction phase analysis and a traditional analysis, the assembled construction model was re-analyzed without completing the construction phase (i.e. it was analyzed in a single step). Figures 4-20 and 4-21 show results obtained the traditional analysis. These can be compared to the results shown in Figures 4-18 and 4-19, respectively. Comparison of the results shows small differences in the stresses. These results indicate that the restraint provided by the structure during construction did not have a significant impact on the effective prestressing force in the members. Based upon these results, the traditional analysis model was used for the service load and ultimate strength analysis.

The effect of the “thin” slab can be seen by comparing the results shown in Figures 4-22 and 4-23. In Figure 4-22, the results from the design (10”) thickness slab are shown with a maximum compressive stress of approximately 850 psi from the prestressing. In Figure 4-23, the results of from the “thin” (8”) slab are shown. The maximum compressive stress increases to approximately 1,100 psi due to the thinner cross-section.

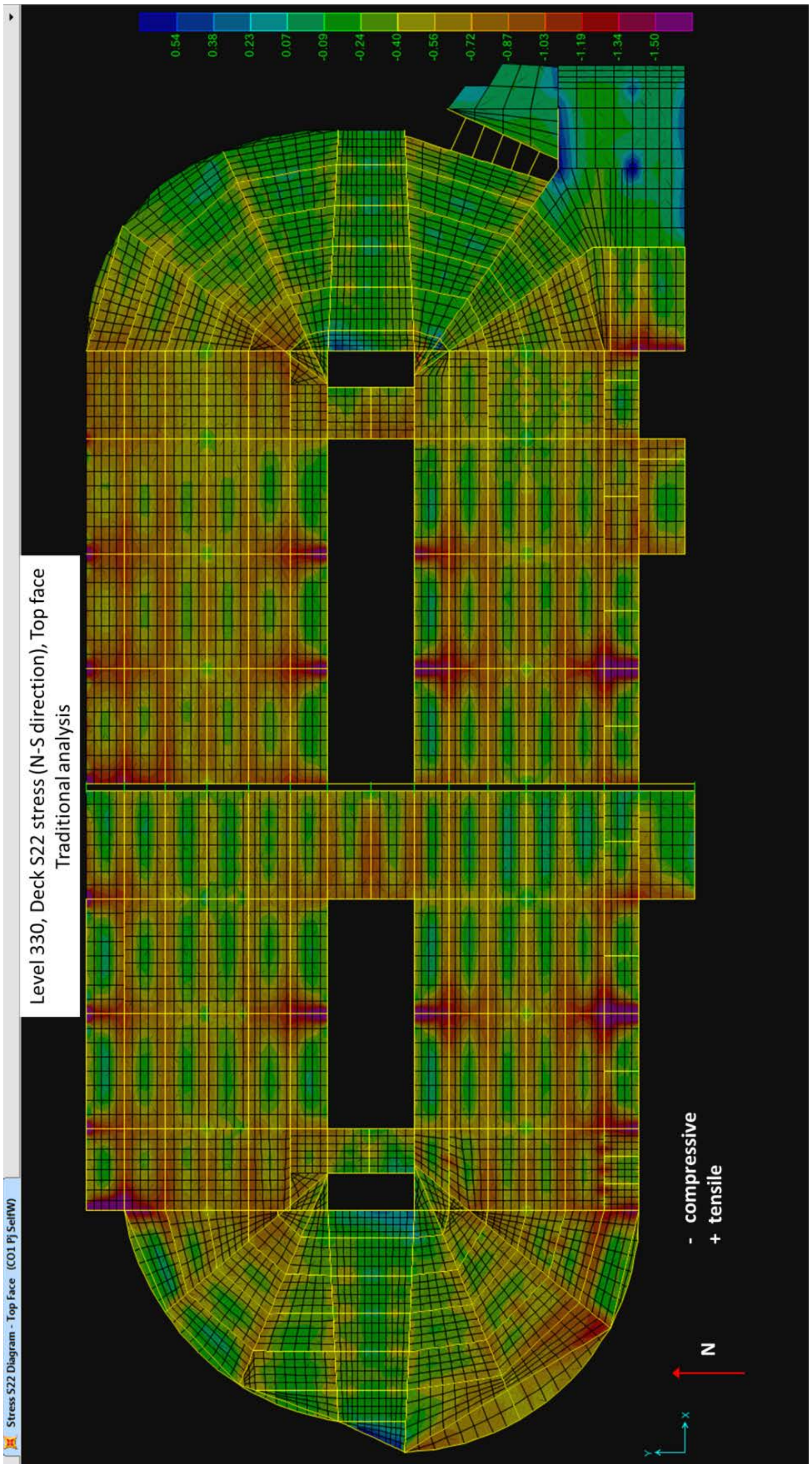

Figure 4-20. Stress contour on Level 330 after construction (traditional analysis), compare to result shown in Figure 4-18.

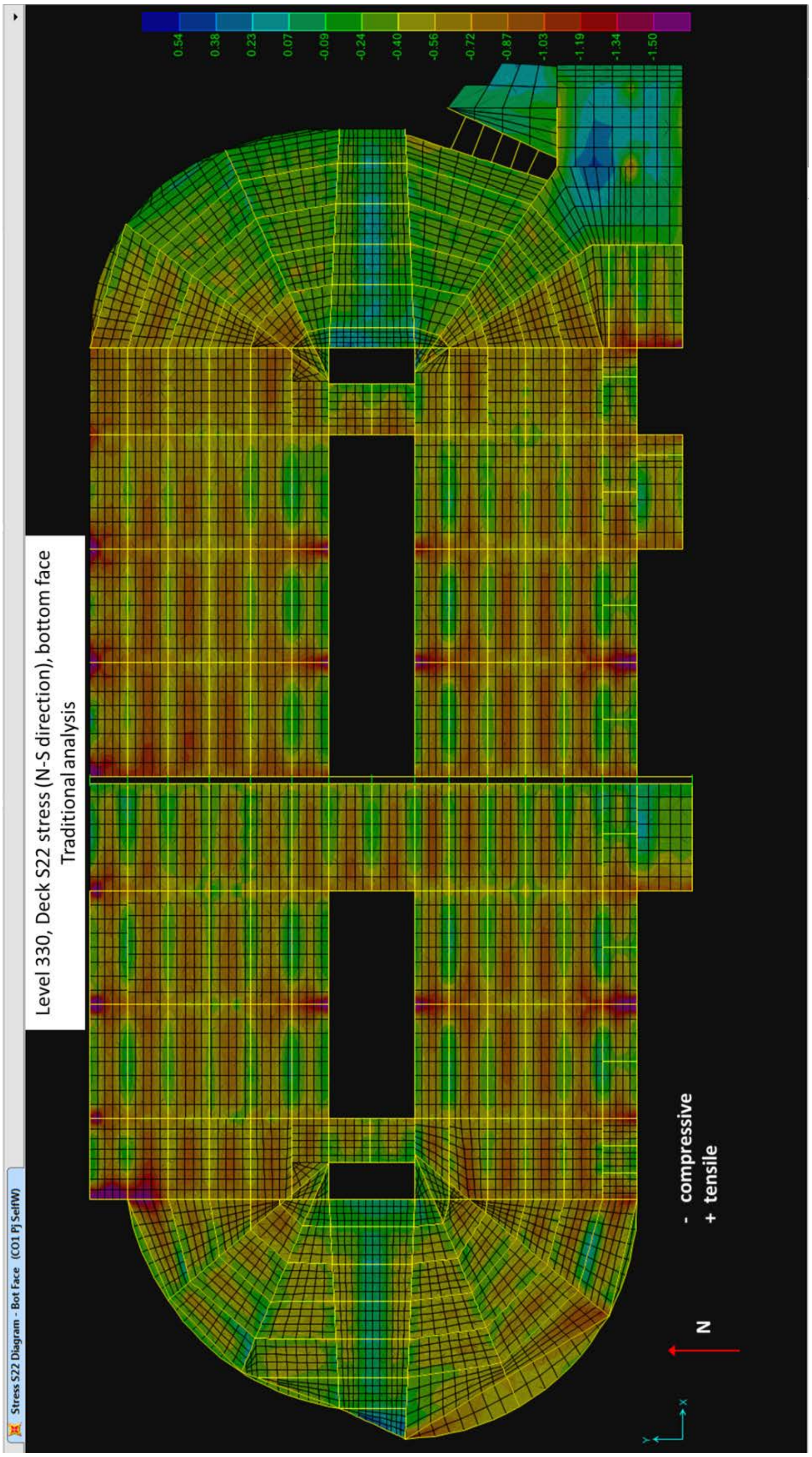

Figure 4-21. Stress contour on Level 330 after construction (traditional analysis) compare to result shown in Figure 4-19.

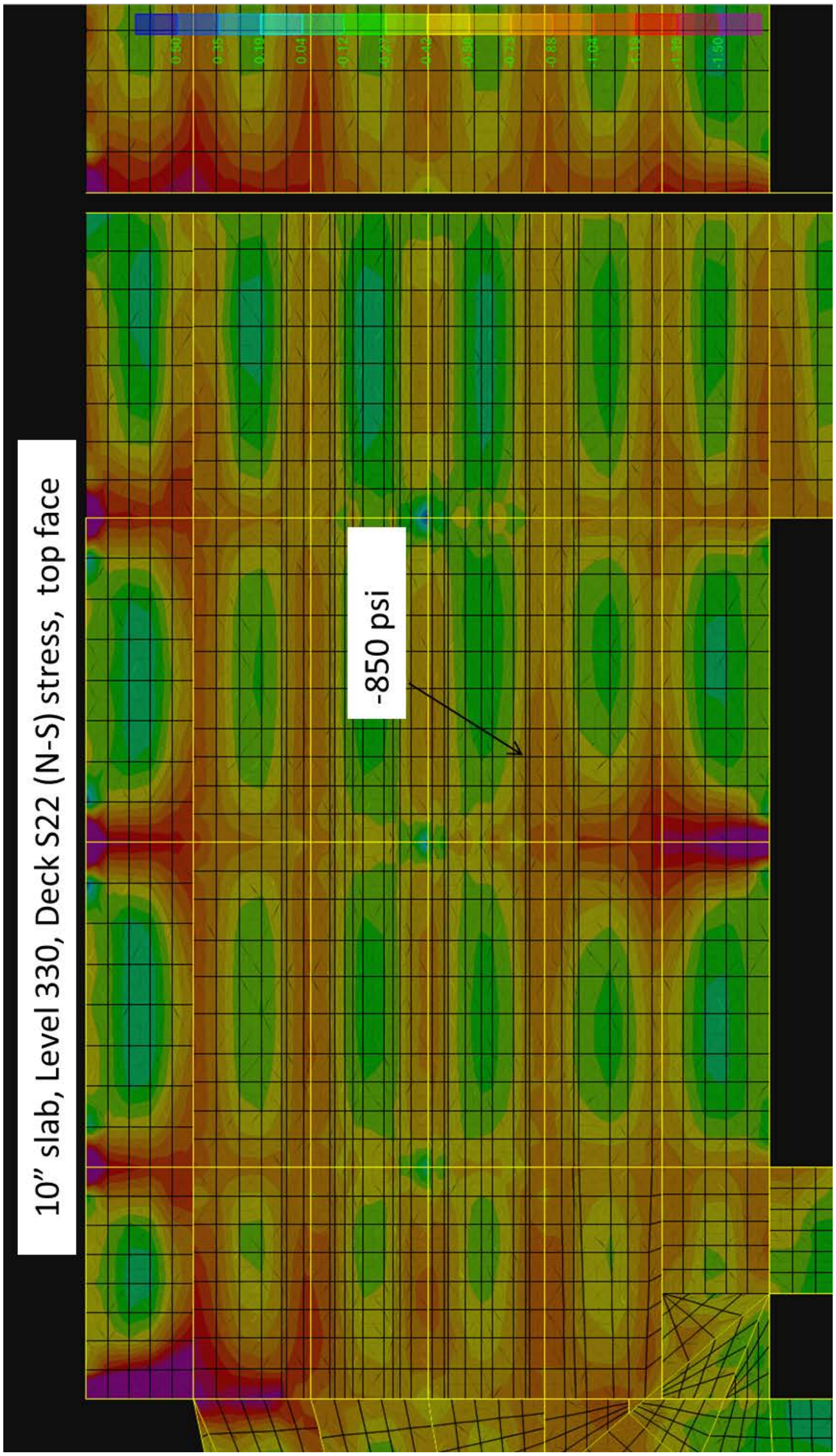

Figure 4-22. Construction phase stress contour - 10" slab.

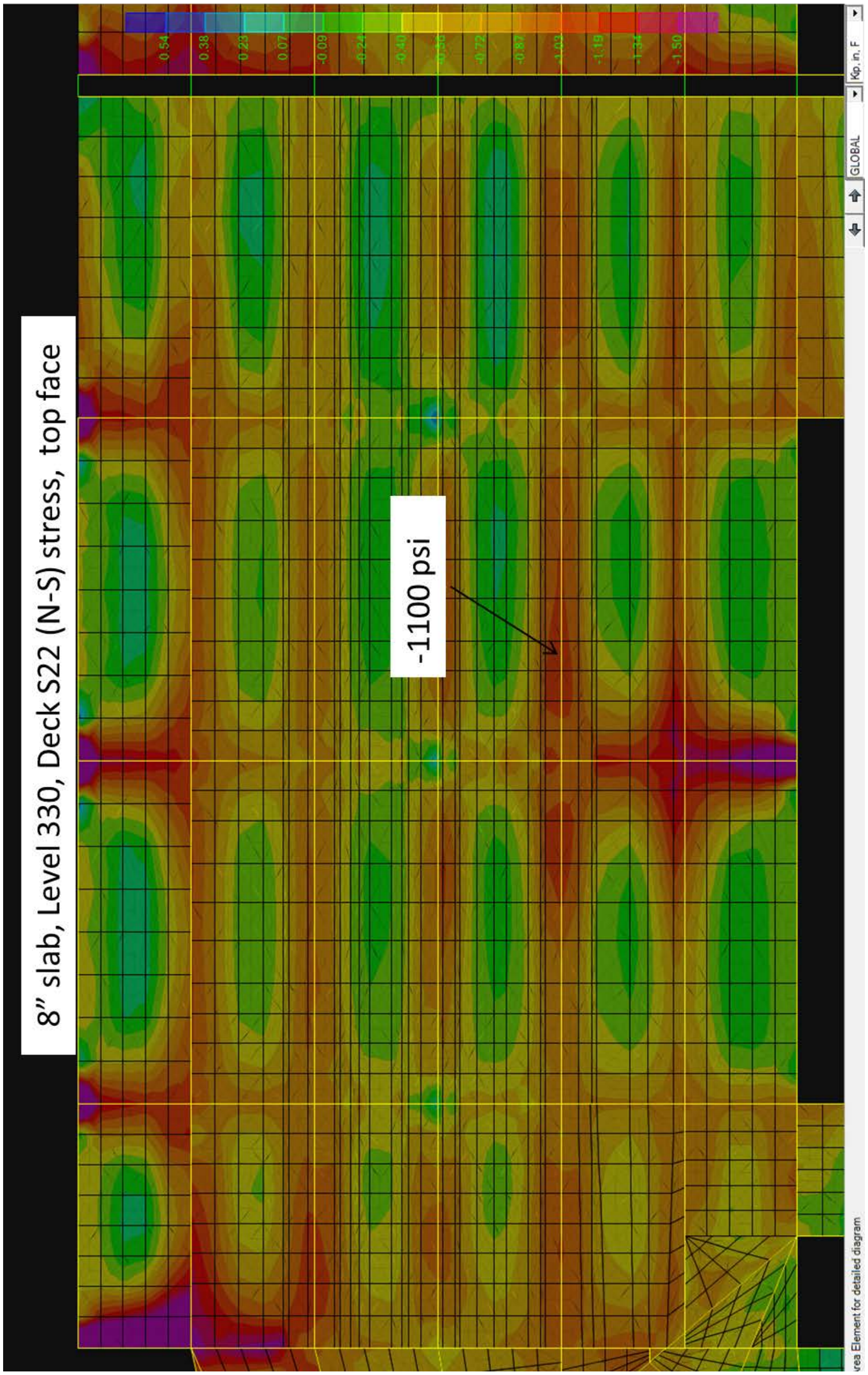

Figure 4-23. Construction phase stress contour - 8" slab.

4.3 SERVICE LOAD ANALYIS

To examine the behavior of the SSTC under service load conditions, a series of analyses were performed with the loadings shown in Figures 4-3 to 4-8. These loadings were developed to determine the capacity of a section of slab on Level 330, which was considered a representative section. The live loading in the analysis included the design vehicles as shown with an impact factor of 33% (design wheel load of 26.6 kips). Two different sets of analyses were performed, one set with the design slab thickness of 10 in., and a second set with the slab thickness reduced to 8 in. in a portion of the structure as shown in Figure 4-11.

4.3.1 Design Slab Thickness Results

The results of the service load analysis indicated that the stresses on top and bottom surfaces of the slabs were less than the allowable stress limits. Representative stress contours from Level 330 are shown in Figures 4-24 and 4-25. The results shown in Figures 4-24 and 4-25 were obtained using the loading shown in Figure 4-3. The peak stresses in the contours show no tensile stresses in the slabs. Based upon the 6 ft. spacing of the loads (wheel spacing) and the 14 ft. beam spacing, the majority of the load is distributed to the stiffer beam elements, this effect is better captured using the three dimensional SAP model compared to the strip analysis performed by PB.

The peak tensile stresses on the bottom of the beams was found to be approximately 700 psi under beam load case 1 (beam load case 1 shown in Figure 4-7). The high stress was limited to the negative moment region of the beams. The stress contour is shown in Figure 4-26. The 700 psi value is in excess of the allowable design stress limit. The peak tensile stress in the girder was found to be 520 psi, which satisfied the allowable stress limits.

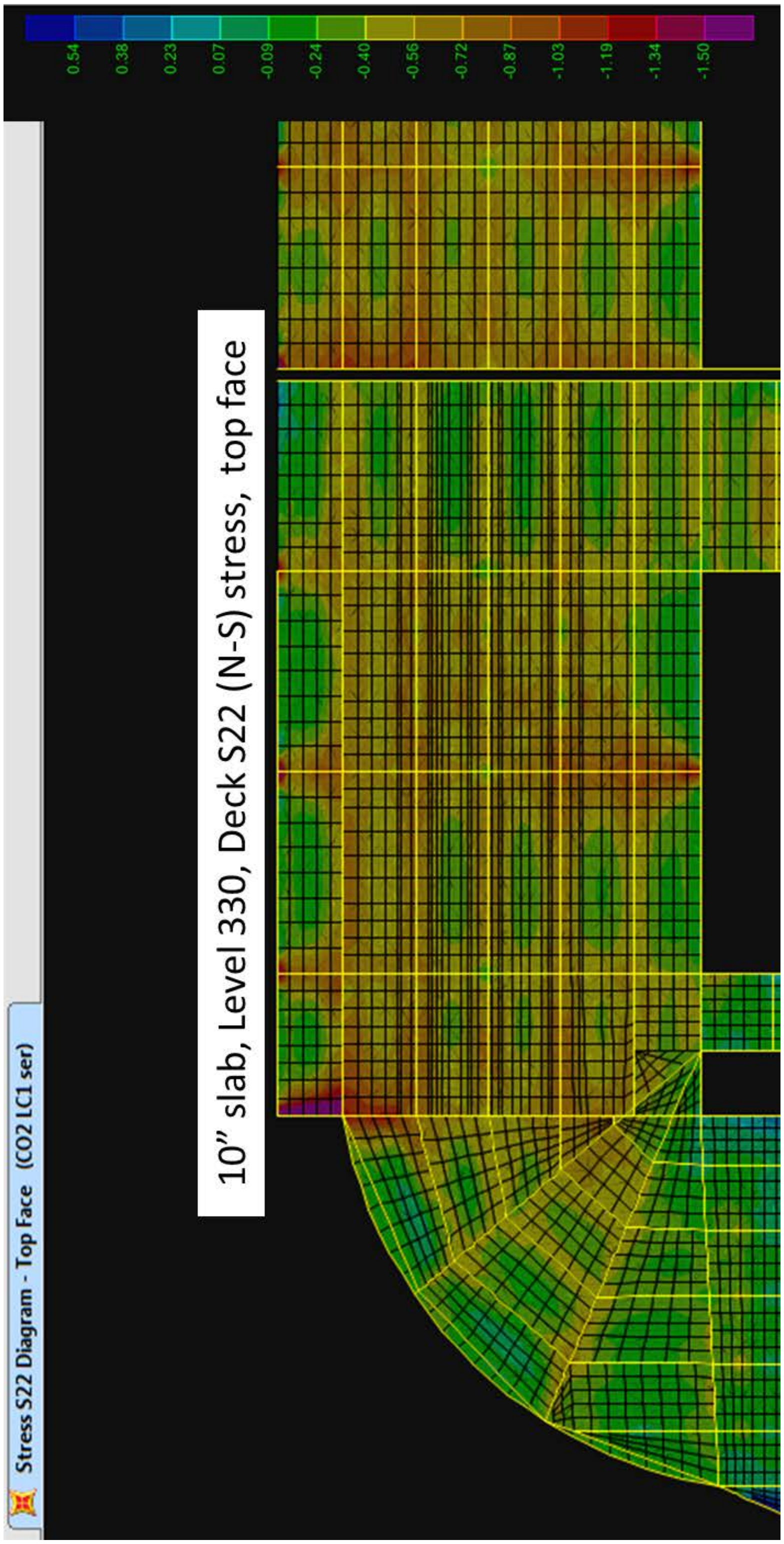

Figure 4-24. Slab analysis results – top surface stress contour.

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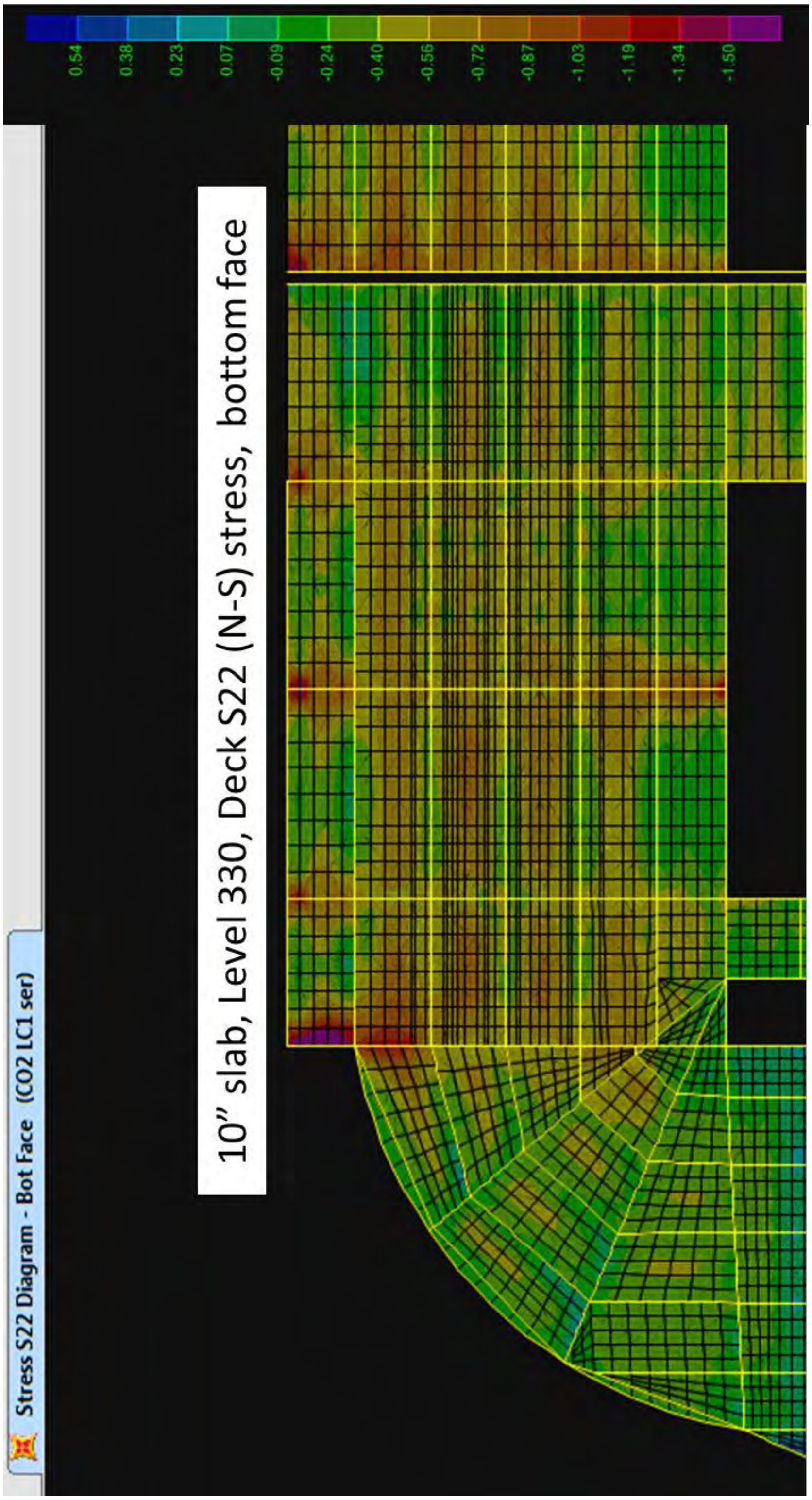

Figure 4-25. Slab analysis results – bottom surface stress contour.

WDP

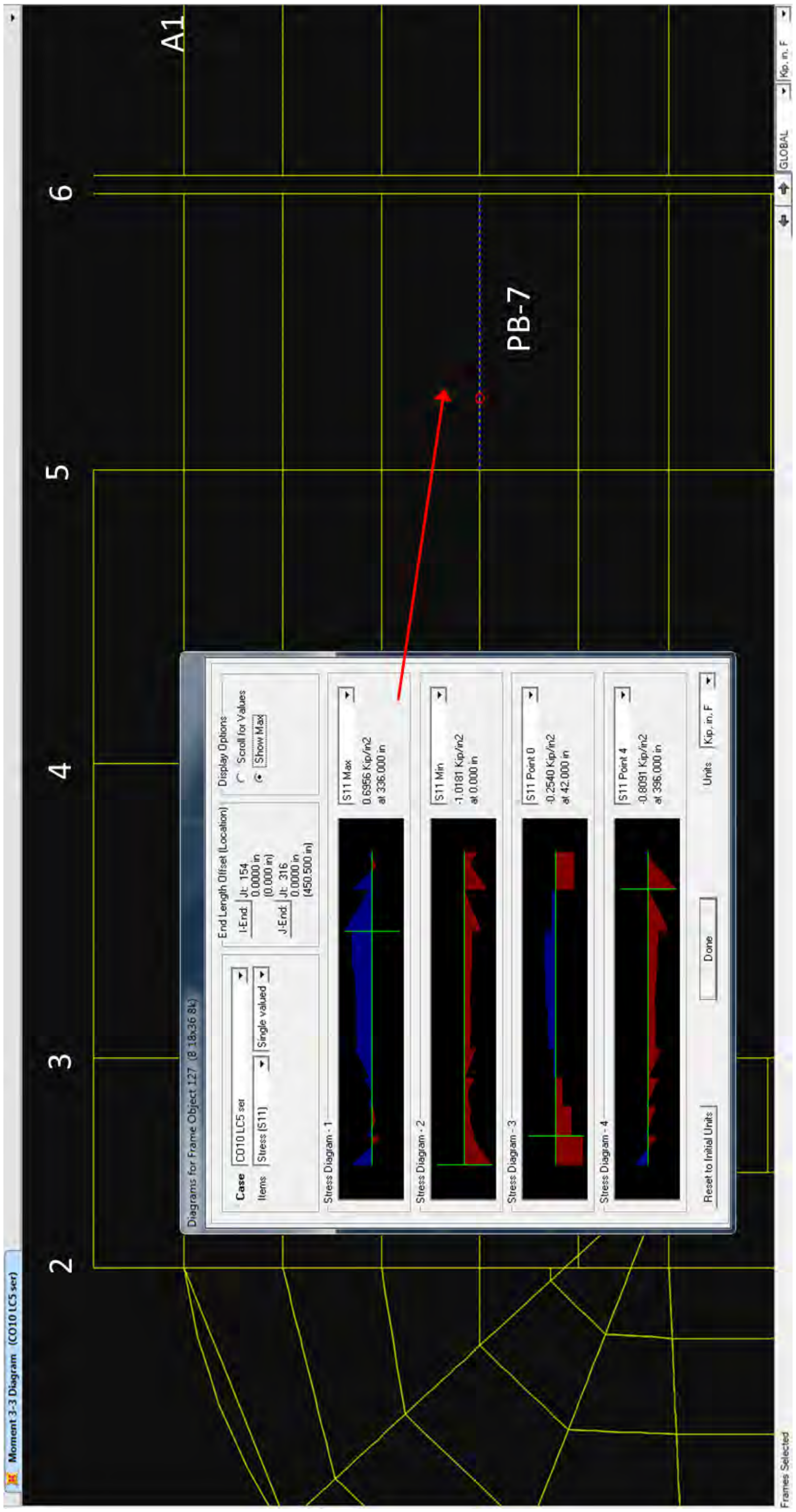

Figure 4-26. Stress contour showing high tensile stress on beam bottom.

WDP

4.3.2 "Thin" Slab Results

The results of the thin slab analysis were generally similar to the results from the analysis of the model with the design slab thickness. Representative stress contours from Level 330 are shown in Figures 4-27 and 4-28. The results shown in Figures 4-27 and 4-28 were obtained using the loading shown in Figure 4-3. The stress limits were within the allowable stress limits shown in Table 4-2. The similar stresses in the thin slab sections were the result of the higher prestressing force in the thin slab (see Figures 4-23 and 4-24). The resulting demands (factored loads) were also similar to the results from the design thickness.

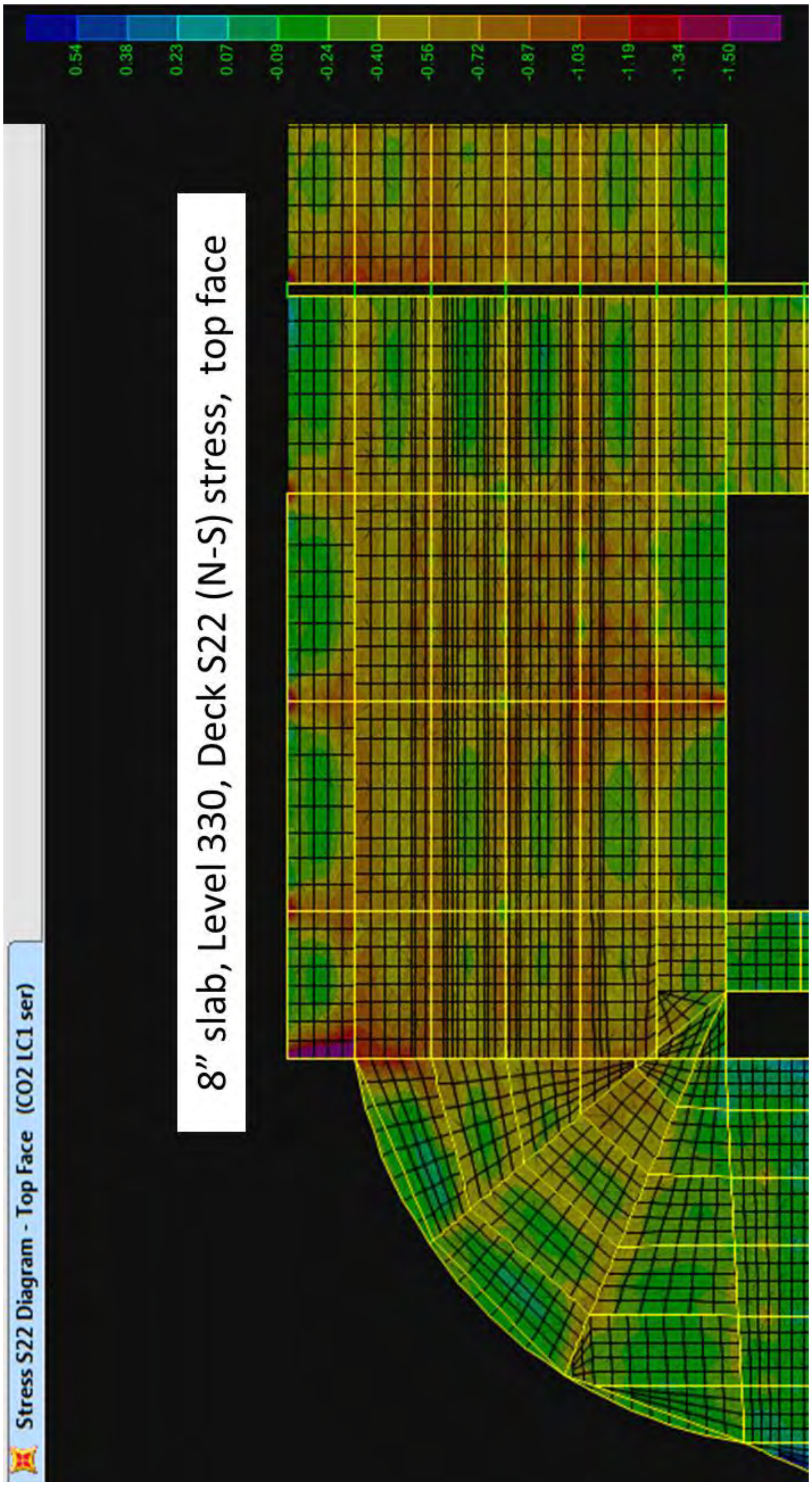

Figure 4-27. "Thin" Slab analysis results – top surface stress contour.

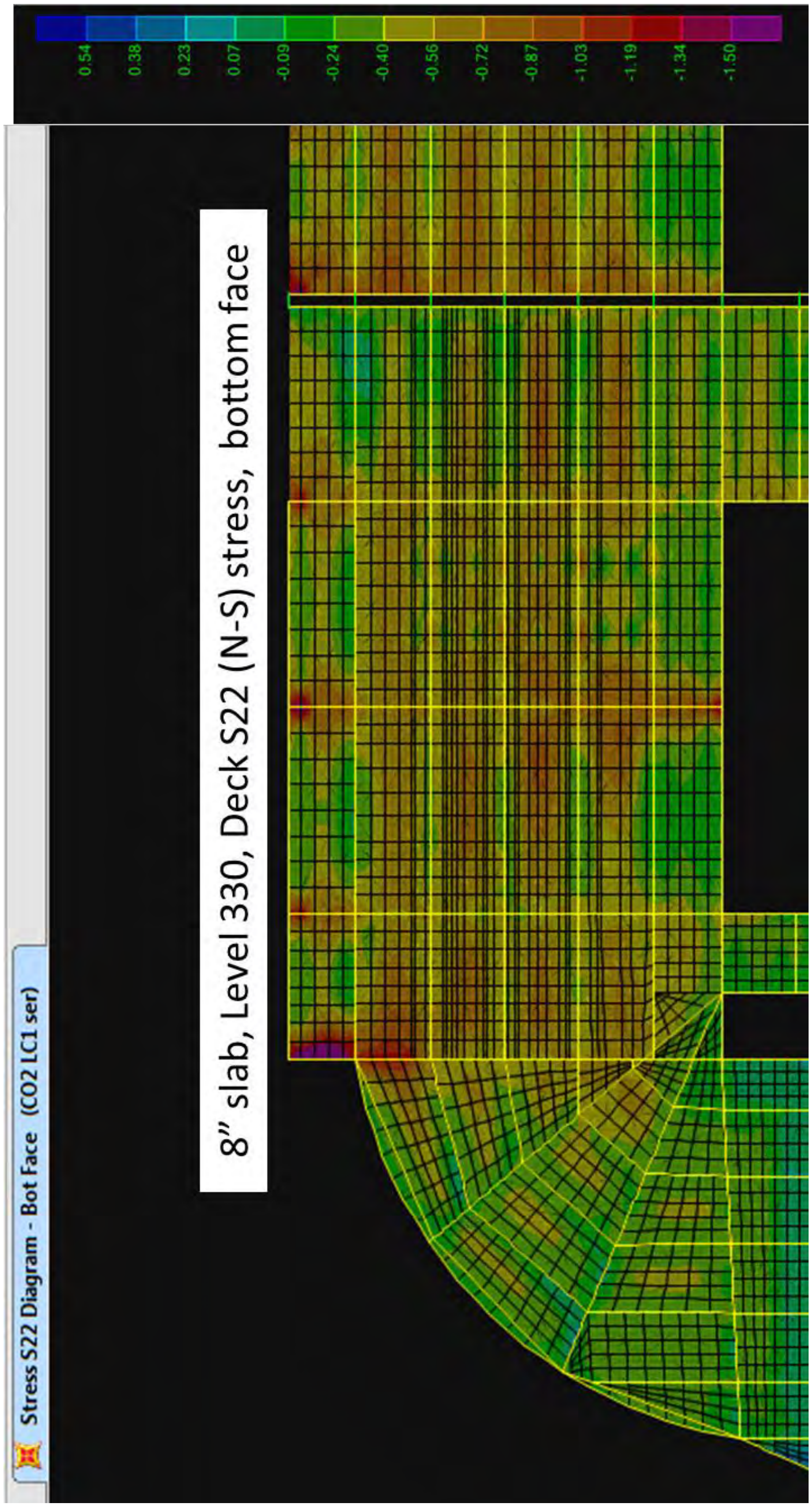

Figure 4-28. “Thin” Slab analysis results – bottom surface stress contour.

4.4 ULTIMATE STRENGTH ANALYSIS

To examine the ultimate strength of the SSTC, the service load analysis was repeated using factored loads. Representative results from the ultimate strength analysis are summarized in Table 4-4. The results for the "thin" member slab analysis were developed assuming a 1 in. reduction in bottom cover and top cover. These results indicate that at the examined sections, the as-designed and as-constructed SSTC has adequate ultimate strength.

Table 4-4 - Summary of Member Demands and Capacities

Member	Demand	Capacity
Slab – 10 in. deep	17 kip-ft /ft	42 kip –ft / ft
	-10.5 / -9.5 kip-ft / ft	-21 / -33 kip-ft / ft
Slab – 8 in. deep	17 kip-ft /ft	31 kip-ft / ft
	-10.5 / -9.5 kip-ft / ft	-15 / -26 kip-ft / ft
Beam PB-6 – 36 in. deep	612 kip-ft	1,815 kip-ft
	-608 / -843 kip-ft	-1,850 / -1,947 kip-ft
Girder PG-8 – 72 in. / 96 in. deep	5,620 kip-ft	20,102 kip-ft
	-11,780 / -7,730 kip-ft	-23,520 / -21,913 kip-ft

4.5 SUMMARY OF ANALYSIS RESULTS

To evaluate the design of the SSTC, a three dimensional structural model was created. The initial models were developed to examine if the phased construction had a significant impact on the effective post-tensioning forces that were developed in the members. The construction stage analysis indicated that the restraint provided by the existing construction had a small effect on the stresses compared to a traditional analysis.

The three dimensional model, created by WDP, included the post-tensioning in the slabs (primary one-way and temperature tendons), beams and girders. Use of a three dimensional model was expected to create a better representation of the flow of forces in the structure compared to strip methods.

To evaluate the response of the structure to design loading conditions, a series of analyses were performed with live loads (HS-25) placed on a section of Level 330.

The resulting stresses and forces on the structure were compared to allowable design stress limits and the calculated capacities of the structure. The results of these analyses confirmed the as-constructed structure had adequate strength.

5.0 LONG-TERM DURABILTY

The long-term durability of a structure is a function of initial construction quality, the extent of routine maintenance performed on structure, and the extent of durability enhancement measures that should be installed on the structure to achieve its design service life. The various evaluations that have been completed at the SSTC have all identified initial construction quality issues that may compromise the long-term durability of the structure. These include:

- Random, map-type cracking on the elevated slab surfaces.
- Through cracks in the beams and slabs that are allowing water leakage through the structure.
- Low entrained air content in the concrete near the slab surfaces.
- Concrete cover that is less than the design requirements.
- Exposed and near-surface post-tensioning tendons

The service environment for the SSTC is considered aggressive with frequent exposure to the freeze-thaw conditions and deicing salts. Accordingly, measures are recommended to improve the long-term durability of the SSTC to achieve its design service life. The following sections describe the causes of the cracking at the SSTC and the expected impact of the cracking on the durability of the structure.

5.1 CAUSES OF OBSERVED CRACKING

The observed cracking at the SSTC can be broadly split into three categories. Cracking which occurred as a result of restrained shrinkage of the concrete, cracking that occurred due to movement of the structure during post-tensioning and map cracking on the concrete slabs likely due to construction finishing operations. The map cracking on the slabs has the greatest potential to adversely affect the long-term durability of the structure and is further discussed in Section 5.2.1.

WDP

The cracks in the beams and columns (see Figures 3-16 to 3-18) at the SSTC appeared to be the result of restrained shrinkage of the structure. Concrete materials begin to shrink after casting as a result of water loss and hydration of the cementitious materials. The post-tensioning of the concrete is intended to pre-compress the concrete to counteract the formation of cracks under service loads. However, the shrinkage of the concrete will begin prior to application of the prestressing. Thus, cracking can occur despite the application of the prestressing. Additional cracking and widening of existing cracks will occur as a result of long-term shrinkage and creep of the concrete.

Cracks were also observed near the base of the columns at the SSTC (see Figures 3-12 to 3-15). These cracks appeared to be result of deformation of the structure during the application of post-tensioning forces to the girders. The post-tensioning of the girders results in elastic shortening of the members and additional long-term shortening due to creep of the concrete. The columns supporting the girders are also displaced, resulting in a bending moment in the columns. The observed cracking in the columns occurred when the tensile strain capacity of the concrete was exceeded. These cracks will widen over time due to long-term creep effects.

Cracking due to restrained shrinkage of the concrete and the post-tensioning are not expected to have a significant impact on the long-term durability of the structure provided water movement through the cracks is mitigated and adequate concrete cover is present.

5.2 DURABILITY CONCERNS

Based upon our site observations, field testing results and the petrographic analysis completed by CTL, the primary factors that will adversely affect the long-term durability of the SSTC are scaling of the slab surfaces and corrosion of embedded reinforcing steel. In addition to damage from these mechanisms, the sidewalks on Level 350 are expected to fail as a result of deterioration from freezing water that cannot escape through the weeps. Replacement of the Level 350 planter waterproofing will also be required during the service life of the structure.

5.2.1 Concrete Surface Scaling

A concrete structure exposed to an aggressive environment including freeze-thaw cycles and deicing salts can sustain scaling of the near-surface concrete. Near-surface concrete with inadequate air content, high water content, high permeability and/or low compressive strength will contribute to scaling vulnerability. Common sign of scaling are microcracking in surface concrete, map cracking, D-cracking and/ aggregate popouts at the surface.

Scaling is a form of surface deterioration that occurs during freezing and thawing of concrete with concentrated salt solutions near the surface. Typically a scaling concrete structure is left with exposed aggregate on the top surface. The key factors that influence scaling resistance in a structure are concrete mixture design (air-entrainment, well-graded and freeze-thaw resistant aggregate, low-permeability mixture design, and optimal water-cementitious material ratio), construction practices (concrete placement, finishing, and curing), and other factors such as concrete compressive strength, and type of exposure such as deicing salt, level of saturation of concrete, and freeze-thaw cycles. Addition of excess water to aid in finishing or failure to allow for evaporation of bleed water at the time of finishing can increase water content in the near surface concrete and make it vulnerable to scaling.

As indicated in the CTL petrographic analysis report, the top surface of the concrete (0 to 30 mm) was found to have a lower entrained air content then the base concrete material in all fourteen of the examined cores. Further, in three of the fourteen cores, the top surface was found to be more water absorbent than the base concrete. An absorbent concrete surface when exposed to moisture is likely to achieve critical saturation. A near-surface concrete with absorbent characteristics and inadequate air content will be highly susceptible to scaling when exposed to freeze-thaw cycles and deicing salt.

Based on the information obtained from the CLT petrographic analysis report, and the visible map cracking on the surface of Levels 330 and 305, the elevated slab surfaces will be vulnerable to scaling when exposed to freeze-thaw conditions and deicing salts.

5.2.2 Reinforcing Steel Corrosion

Reinforcing steel in concrete is protected from corrosion by a high pH passivating layer that forms on the surface of the steel. Corrosion of reinforcing steel occurs when sufficient water and oxygen are present at the level of the reinforcing steel to overcome the passivation of the steel. The presence of chlorides in the concrete will act to break down the passivating layer, accelerating the corrosion process. To protect reinforcing steel from corrosion, combinations of concrete cover and coatings on the reinforcing steel are used to prevent the ingress of water and chlorides to the level of the reinforcing steel.

The various investigations into the construction of the SSTC have indicated that the cover to the reinforcing steel is lower than required by the design documents in various parts of the structure. The uncoated reinforcing steel with low cover (see Figure 3-19) will be the most susceptible to future corrosion. The reinforcing steel in the elevated slabs is epoxy coated, and therefore, will be more resistant to corrosion. Based upon the limited concrete cover and observed cracking, uncoated reinforcing steel corrosion is likely to occur at the SSTC at some point within the service life of the structure.

5.2.3 Thermal Movement Accommodation

A major issue with the design of the SSTC was the lack of details in the structure to accommodate movement due to thermal changes. As a result, the significant cracking has occurred as a result of early age (drying) shrinkage of the concrete and during normal thermal movement. The most significant restraint is provided by the foundation walls in the northeast corner of the structure, accordingly the most extensive cracking in the beams is located in this area. Cracking that has occurred as a result of restrained drying shrinkage is shown in Figures 3-16 and 3-17.

On the elevated slabs, the beams and girders members also provide a significant amount of restraint to shrinkage of the concrete in the elevated slabs. The restraint of the slabs is a significant contributing factor to the widespread cracking observed in the slab elements.

5.3 ROUTINE MAINTENANCE

The long-term integrity of any exposed structure will be enhanced by routine maintenance. The primary goal of the maintenance is the removal of deicing salts from the structure. Ingress of chlorides will act to accelerate reinforcing steel corrosion and promote surface scaling. For the SSTC recommended routing maintenance items include:

- Semi-annual power washing of elevated slabs to remove dirt, oil stains and chlorides.
- Removal of debris from expansion joints.
- Cleaning and removal of debris from drains.
- Sealing of any newly formed cracks.
- Replacement of crack filler materials.
- Replacement of expansion joints.
- Restriping of travel lanes.
- Landscaping upgrades.

Completion of the routine maintenance will be required regardless of the strategies adopted for repairs at the SSTC.

5.4 LIFE CYCLE COSTS

Based upon the design and construction deficiencies, it is unlikely that the 50 year design service life will be achieved without a strategy of long-term protection for the exposed elements of the structure. The life cycle cost associated with these repairs is difficult to determine. To estimate the life cycle costs associated with repairs to maintain the structure during a 50 year service life, the following costs need to be considered:

- Cost for repair to address known and hidden construction defects.
- Costs for future replacement of repairs and costs for additional concrete repair area in the future.
- Costs associated with installation of a long-term protection strategy for the elevated slabs.
- Costs associated with future replacement of the selected long-term protection strategy for the elevated parking slabs.
- Costs for design and management of future repairs.
- Additional costs associated with escalation of repair prices during the service life of the structure.
- Premium costs for completion of repairs during periods of limited operation of the facility.
- Direct costs associated with loss of use of the facility during repair periods.
- In-direct costs associated with loss of use of the facility during repair periods, such as cost for redirecting buses, changing traffic patterns, additional traffic control and increased street congestion.
- In-direct cost associated with operation of a structure perceived to be in a constant or near-constant state of repair.

All of the costs listed above are variable and therefore determination of the life cycle cost associated with the facility is difficult to determine. However, it is clear that these costs will significantly higher than the cost associated with a properly designed and constructed facility.

6.0 SUMMARY AND RECOMMENDATIONS

An evaluation of the Silver Spring Transit Center has been completed. The goal of the evaluation by WDP was to assess both the structural design and expected long-term durability of the structure prior to the acceptance of the structure into the WMATA inventory. The evaluation was motivated by the observation of concrete cracking and questions regarding the thickness of the concrete slabs in the elevated portions of the structure.

WDP

To examine the structure, WDP performed a series of nondestructive tests to evaluate the as-built condition of the structure. These tests confirmed that sections of the elevated slabs were significantly thinner than required by the design documents and were outside of the specified construction tolerances. Additional conclusions from the field evaluation by WDP included:

- The grout used in the bonded post-tensioned tendons did not contain chlorides.

- The extent of cracking on the elevated slabs had increased compared to the previous surveys by PB.
- The resistivity of the concrete was high, which indicates the concrete, outside of the cracked areas will be resistant to chloride ion penetration.

Based upon the previous evaluation reports and our site observations, numerous construction defects are present in the structure. These defects include:

- Omission of the post-tensioning tendons in the Level 330 pour strips, which will necessitate replacement of the pour strips.
- Thin slabs in portions of Levels 330 and 350.
- Exposed and low concrete cover to post-tensioning tendons on the slab surfaces.
- Low entrained air content on the top section of the elevated concrete slabs.
- Extensive cracking on the elevated slab surfaces.
- Lack of details in the construction plans and construction procedures to accommodate normal thermal movements in the structure.

A strength evaluation of the as-designed and as-built structure by WDP indicated that the overall structural design of the structure was adequate. In isolated areas, the allowable stress limits were exceeded. The evaluation was based upon an analysis of representative sections and does not represent a complete review of the design. The primary design defect was the inability of the structure to accommodate normal thermal movements due to the significant restraint provided by the structural framing.

Despite the generally adequate structural design, the significant construction deficiencies have resulted in a structure that is unlikely to achieve the 50 year design service life specified in the WMATA design without implementation of a long-term protection program. The construction deficiencies include omission of post-tensioning tendons in the pour strips on Level 330, low cover to post-tensioning tendon ducts at several locations, low entrained air content in the top surface of the elevated slabs and extensive cracking on the slab surface as a result of both restrained shrinkage and finishing problems.

Based upon the extensive design and construction deficiencies, the SSTC will experience significantly higher costs for maintenance, future repairs and loss of use during repair periods compared to a properly designed and constructed facility. As-built, the SSTC is not expected to achieve its 50 year design service life without significant repairs to improve the durability of the structure.